# Review of Side Friction Factors in Highway Curve Design of Higher Speed Freeways


Jaisung CHOI
Professor
Department of Transportation Engineering
University of Seoul
90 Jeonnong-dong, Dongdaemun-gu
Seoul, South Korea
Fax: +82-2-2210-2653
E-mail: traffic@uos.ac.kr

Sangyoup KIM
Graduate Student
Department of Transportation Engineering
University of Seoul
90 Jeonnong-dong, Dongdaemun-gu
Seoul, South Korea
Fax: +82-2-2210-2653
E-mail: road@uos.ac.kr



**Abstract:** This paper presents a review of the conventional curve design theory and its applicability to SMART highway that is a new type of freeway with higher design speed of 130-160 km per hour. The highest design speed is now 120 km/h, and its increase calls for a safety check for both operating vehicles and motorists around horizontal curves. Currently, this check is done by comparing the supplied friction values against the required values. The assumption in this check is that vehicles and motorists are safe from having vehicle sliding if the supplied exceeds the required. Whereas considered satisfied for current freeway design speed levels, this assumption is not yet screened for 130-160 km/h speed levels. This research made vehicle speed measurements in three representative horizontal curves with relatively long curve radii in flat terrains located in West Sealine Freeway. The measurements revealed that based on AASHTO design standard motorists seem to experience driving discomfort when negotiating curves at higher than 130 km/h speeds. This finding is informative, and it is recommended that highway engineers pay extra attention when they design this type of curve in future SMART highway.



***Key Words:*** *higher design speeds, horizontal curve design, side friction factors, the required, the supplied*


## 1. INTRODUCTION

### 1.1 Background and objectives of the study

Presently a national project of building SMART highway that is a new type of freeway with higher design speed of 130-160 km per hour is underway. SMART highway building and particularly increasing design speed implies motorist's faster driving, and this concerns the public including the law-makers and academics because the maximum design speed in current highway geometric design standards is 120 km per hour that is well below the proposed SMART highway design speed. Motorist safety particularly involves checking the appropriateness of horizontal alignment design standards because horizontal alignment varies more frequently than vertical alignment or cross-section does to reflect surrounding highway conditions. This research thus is limited to reviewing current horizontal alignment standards.

Differently from tangent sections, horizontal curve design theory involves how to deal with centrifugal force generated by fast moving vehicles on the curve. Also, the size of the centrifugal force increases with shorter curves. Therefore, for the safety and comfort of driving motorists, the current design method sets the minimum curve radii for a practical range of highway design speed and requires engineers to apply larger radius than the minimum (8). In the minimum radius calculation, design variables including vehicle operating speed, superelevation, and side friction factor are considered, but among them side friction factor is prevailing. In fact, there are many research findings available for understanding side friction and its characteristics in curve design. For example, Meyer (1949), Barnett (HRB 1936), Moyer & Berry (HRB 1940), and Stonex and Noble (HRB 1940) have published their research findings on side friction values and vehicle speeds made mostly in the US. However, these studies were made in early days of highway design theory development, and it is critical that these research findings, which were based on design speed of 120 km/h at the highest, are not directly applicable to SMART highway design. For instance, J. Emmerson (1969) has calculated actual side friction factors observed at six horizontal curve sites based on passenger car speed data, and found that at the curve radius range of 196-350 m the average value of side friction factors indicated 0.11 with more than 80 % passenger cars having less than 0.15. In contrast, in the case of curve radius range of 70-330 ft, side friction factors were observed to be 0.27 and 0.22 with more than 90 % passenger cars having greater than 0.15. This indicates that observed side friction factors can be different from theoretical values (2). McLean also supported this argument with his finding that motorists experienced higher side friction on curves (7). In summary, it is not desirable to apply the current highway design standards to SMART highway design, and this research has done a review of horizontal alignment design theory and discussed the expected variations of design variables associated with increasing design speed in SMART highway. A field study analysis by measuring vehicle speeds at relatively flat curves in a freeway segment was done to investigate the appropriateness of highway design standards.

The followings are the research objectives:

1. To check the appropriateness of applying current horizontal curve design theory for SMART highway
2. To measure speed levels at very flat curves and calculate actual side friction levels in existing freeways to make a comparison with theoretical values

### 1.2 Research Scope and Approach

This research has used the following approach. First, existing curve design procedures and their side friction factor values were reviewed for 18 countries over the world to analyze their expected variations at higher speed freeways whose design speed is 130-160 km per hour. Second, to characterize side friction factors at horizontal curves in the higher speed freeways, this research has made a substitution and selected very flat horizontal curves in an existing freeway in West Sealine Freeways in South Korea, and made vehicle speed observations on the curves. Figure 1 illustrates the research approach.

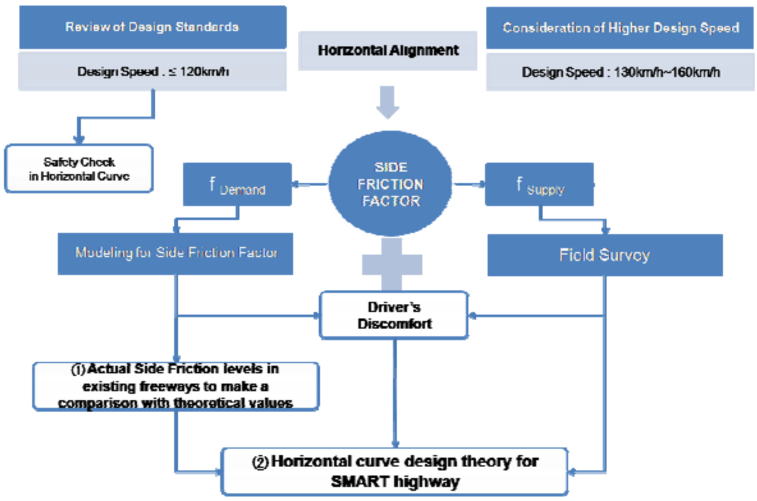
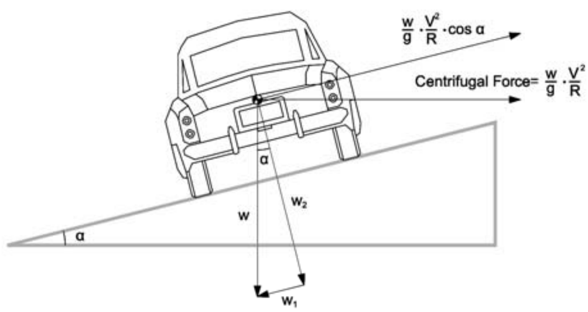



Figure 1 Research Approach

## 2. REVIEW OF SIDE FRICTION FACTORS IN HORIZONTAL CURVE DESIGN

### 2.1 Mass Point Formula

When a vehicle moves in a circular path, it is forced radially outward by centrifugal force. To counterbalance the force and stay moving in the circular path, the friction force that is developed by vehicle weight and friction factor between tires and pavement must be greater than the centrifugal force. In highway geometric design practice, engineers use superelevation for a long time to supplement the friction force and facilitate smoother vehicle travel in a curve (1). In the design of highway curves, there exists the relation between design speed and curvature and also the joint relations with superelevation and side friction (1). And this relation is called the Mass Point formula in highway design.




$$w_1 = w \cdot \sin\alpha \quad \text{(Force caused by Superelevation)}$$

$$w_2 = w \cdot \cos\alpha \rightarrow f \cdot w \cdot \cos\alpha \quad \text{(Side Friction)}$$

$$\frac{w}{g}\cdot\frac{V^2}{R}\cdot\cos\alpha \leq f \cdot w \cdot \cos\alpha + w \cdot \sin\alpha$$

$$\frac{V^2}{g \cdot R} \leq f + e \quad (e = \tan\alpha)$$

Figure 2 Centrifugal Force in Horizontal Curve

## 2.2 Application of Side Friction Factors

When a vehicle travels and an outside force which acts to the vehicle in perpendicular direction can generate the friction force. In highway curve design this perpendicular force is the centrifugal force discussed in the previous section and the reacting force is the side sliding friction force. The ratio of the side sliding friction force and normal force is called as the side sliding friction factor or the side friction factor in highway curve design. Based on an international survey for the use and measurements of the side sliding friction factor (6,7,8), this value varies depending upon pavement texture and wearing condition and shows: Asphalt cement concrete (0.4-0.8), Cement concrete (0.4-0.6), and icy condition (0.2-0.3).

In highway curve design, there are two design criteria for determining suitable design levels of the side friction factor. First, does the vehicle fail to have a proper side friction level and actually slide on the curve? Second, do motorists feel intolerable levels of discomfort due to centrifugal force while driving the curve? These two side friction factors can have different values, and highway curve design usually adopts the value based on the second method. To determine them, the US and some other nations have made numerous measurements and tests, and Figure 5 summarizes their results (1).

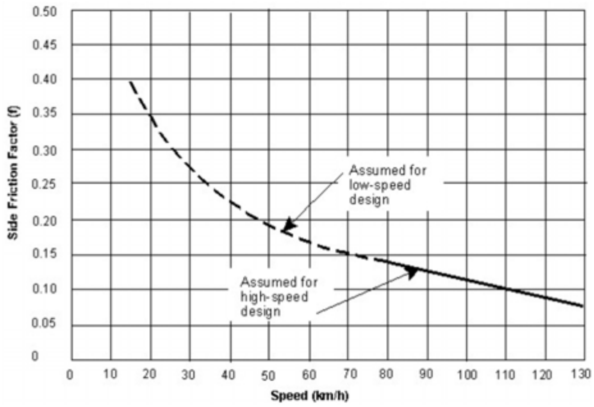



Figure 3 Side Friction Factors for Highway Curve Design

Interestingly, there recently was an initial stage research for establishing highway design standards to be used in higher design speed freeways in the US, and its conclusion included the side friction levels of 0.08-0.04 at the increased freeway speed (3). The authors reviewed this research report and found that they simply utilized the existing relationship between side friction factor and design speed (3). A different approach for measuring side friction factors was adopted in Europe. In Switzerland, 300 sites were selected and a relationship expressing their pavement conditions and side friction force at different speed levels were developed. And based on the relationship two side friction factor equations, one for the longitudinal direction and another for the side direction, were published (8). McLean in Australia asserted based on his empirical analysis that the side friction values specified in AASHTO design guideline were too low. Then he proposed to increase the current side friction values ranging 0.11-0.19 to 0.08-0.35 (7). Meanwhile a group of researchers in Germany including R. Lamm (1999) investigated several European country cases and proposed a new relationship for finding relevant side friction factors in highway curve design as shown in Eqn. (2).

$$f_R = 0.27 - 2.19 \times 10^{-3} V_d + 5.79 \times 10^{-6} (V_d)^2 \qquad (2)$$

Where, $f_R$ : Side Friction Factor

$V_d$ : Design Speed (km/h)

Table 1 Summarize of International Practices of Side Friction Factors in Curve Design

| Design Speed (km/h) | Austria | Belgium | Canada | France | Germany | Ireland | Italy | S Korea |
|---|---|---|---|---|---|---|---|---|
| 50 | | | 0.16 | | | | | 0.16 |
| 60 | 0.16 | 0.16 | 0.15 | 0.16 | 0.14 | 0.15 | 0.17 | 0.14 |
| 70 | 0.15 | | 0.15 | | 0.12 | | | 0.13 |
| 80 | 0.14 | 0.13 | 0.14 | 0.13 | 0.11 | 0.14 | 0.13 | 0.12 |
| 90 | 0.13 | 0.11 | 0.13 | | 0.10 | | | 0.11 |
| 100 | 0.11 | | 0.12 | 0.11 | 0.09 | 0.13 | 0.11 | 0.11 |
| 110 | | | 0.10 | | | | | 0.10 |
| 120 | 0.10 | 0.10 | 0.09 | 0.10 | 0.07 | 0.12 | 0.10 | 0.10 |
| **Design Speed (km/h)** | **Luxem bourg** | **Nether land** | **Portugal** | **South Africa** | **Spain** | **Sweden** | **Switzer land** | **U.S** |
| 50 | | | | 0.16 | | 0.18 | 0.19 | 0.16 |
| 60 | 0.16 | 0.17 | 0.16 | 0.15 | 0.16 | | 0.16 | 0.15 |
| 70 | 0.15 | 0.15 | 0.15 | 0.15 | 0.15 | 0.15 | 0.15 | 0.14 |
| 80 | 0.14 | | 0.14 | 0.14 | 0.14 | | 0.14 | 0.14 |
| 90 | 0.13 | | | 0.13 | 0.14 | 0.12 | 0.13 | 0.13 |
| 100 | 0.12 | 0.12 | 0.12 | 0.13 | 0.13 | | 0.12 | 0.12 |
| 110 | 0.11 | | | 0.12 | | 0.10 | 0.11 | 0.11 |
| 120 | 0.10 | 0.08 | 0.10 | 0.11 | 0.10 | | 0.10 | 0.09 |

## 3. FIELD STUDY OF SIDE FRICTION FACTORS

At present higher design freeways such as SMART highway do not exist in S Korea. Therefore, to characterize vehicle operating in horizontal curves for higher design freeways, this research selected the most favorable highway sections available in this country and made speed measurements on the selected sections. The selected sections are located in West Sealine Freeway within Kwanchun-Dechun and Buan-Julpo cities. There were 3 horizontal curves in the selected sections with curve radii greater than 2,000 meters and vertical grades of -3 to +3 %. The speed survey was made by subdividing the curves into three different segments and applying both the spot speed measurement and license plate technique. The average speed values were finally utilized as the vehicle speed. In the meantime, this research investigated the as-built plan and profile drawings for each site to get curve radius and superelevation data. Using these field study information and Mass-Point formula explained previously, this research could obtain the available side friction levels experienced by drivers on the curves. Finally the appropriateness of AASHTO design guideline as to side friction factors was reviewed by plotting each side friction values in one drawing as shown in Figure 4. In Figure 4, the horizontal axis indicates the ascending order placement of the speeds measured in this field study with N being the total sample size.

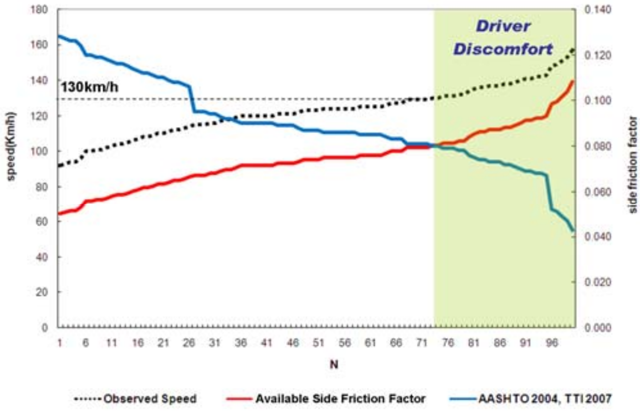



Figure 4 Comparison of $f_{RM}$ with $f_{RR}$

The result revealed that actual side friction level experienced by drivers during curve driving would increase with higher speeds. In contrast, in Table 1, the side friction factors, which are maximum side friction factors and developed to promote driver comfort during curve driving, specified in the current highway curve design show a decreasing pattern with speed increase. Therefore, by overlapping these two side friction factors that were obtained by two different methods, one by AASHTO and the other by field measurement in this research, one can be aware of the followings:

- Although the horizontal curves radii in the field study sites were greater than 2 km, measured side friction factors seem to violate AASHTO design side friction factors when motorists driving faster than 130 km/h speed level.
- For some reason, it was found that the superelevation values in the selected curves involved Normal Crown that had negative superelevation values.

Thus, to resolve these problems, the research proposes two approaches. First, a new set of side friction factors being relatively higher than the current values should be developed. This is because even though the natural setting of S Korea is far too different from the one in the US, the current side friction values generally follow the US value. This research decided to look into worldwide practices particularly including European nations, and attempted to develop more reasonable values for highway curve design.

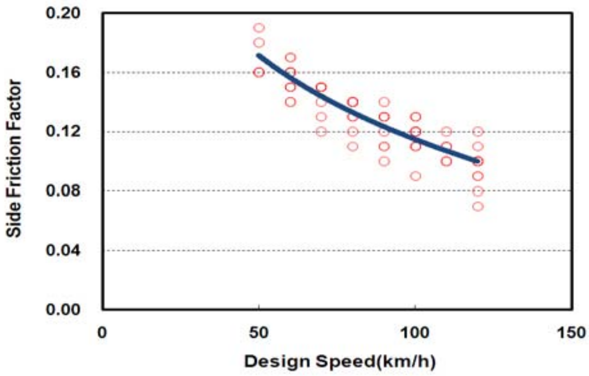



Figure 5 Plotting of Various Side Friction Factors with Design Speed

And the research made an assumption that the model relating side friction factors and design speed would be a logarithmic function. Applying SPSS to the international side friction factors, Eqn. (3) was obtained.

$$f_R^{'} = 0.49 - 0.08 \times \ln(V_d) \tag{3}$$

Where, $f_R^{'}$ : Side friction factor for this research

$V_d$ : Design speed (km/h)

Table 2 summarizes the statistics of Eqn. (3).

Table 2 Model Summary of Eqn. (3)

| Confidence Level | R | $R^2$ | Adjust R2 | Standard Error | t-Statistics | |
|---|---|---|---|---|---|---|
| | | | | | Vd | Constant |
| 95% | 0.905 | 0.819 | 0.817 | 0.010 | -20.401 (0.000) | 27.685 (0.000) |

Multiple correlation coefficient and determination coefficient indicated 0.905 and 0.819, respectively. The regression equation involved a significantly high power of explanation. Also, t-statistics indicated high significance. Table 3 shows the summary of the analysis of variance, and with F value of 416.21 the regression equation obtained in this research had high significance in explaining the sum of squares for independent variables.

Table 3 Analysis of Variance of Eqn. (3)

| | d.f | Sum of Squares | Mean Squares | F | Sig. F |
|---|---|---|---|---|---|
| Regression | 1 | 0.0440 | 0.0440 | 416.21 | 0.000 |
| Residual | 92 | 0.0097 | 0.0001 | | |
| Total | 93 | 0.0537 | | | |

With this newly developed regression equation of Eqn. (3), side friction factors for different design speeds were calculated and compared with other values including AASHTO, Switzerland, and R. Lamm. Figure 6 exhibits the comparison result.

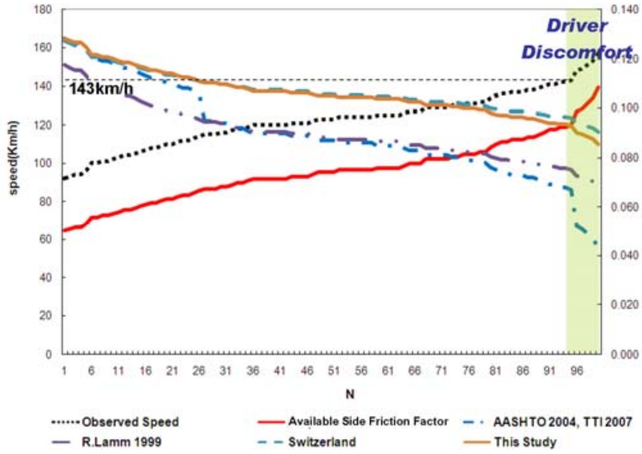



Figure 6 Comparison of $f_{RM}$ with  Revised Side Friction Factors

Applying Eqn. (3), this research discovered that driver discomfort now started to occur at 143 km/h speed level. This speed is 13 km/h higher than the previously found driver discomfort speed level, and it is concluded that in higher speed freeway design a new set of side friction factors, which reflect worldwide highway curve design practice, should be considered.

Next, this research is concerned with superelevation values for the observed sites. For some reason, 0 % superelevation value was applied and observed so in the curves. And this was confirmed in this research by double checking of as-built plan and profile drawings.  This is not correct, and this research proposes proper treatments. Applying Eqn. (3), this research determined that whereas with 0% superelevation, driver discomfort occur if driving faster than 152 km/h, with proper superelvation design driver discomfort starts only when driving faster than 160 km/h. This implies that proper superelevation application is such an important design element that engineers should pay extra attention in its application in higher speed freeway design.

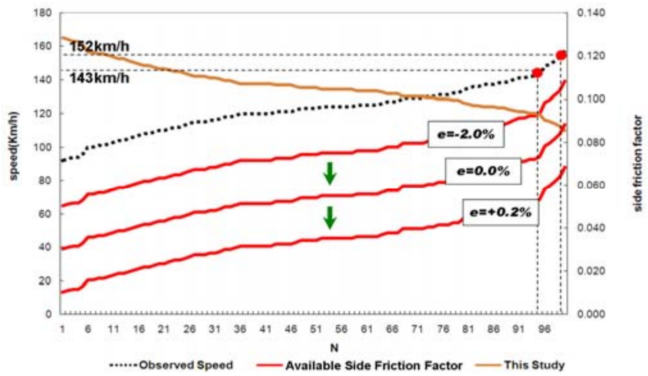



Figure 7 Variation of Driver Discomfort in Highway Curves by Superelevation Values

## 4. CONCLUSION

In this research the existing curve design procedures and their observed side friction factor values in selected field sites in South Korea were reviewed for preparing to build higher speed freeways. And the followings were found in this research:

- Although the horizontal curves radii in the field study sites were very large with more than 2 km, actual measuring vehicle speeds and calculating side friction factors in this research revealed that applying AASHTO design side friction factors in South Korea may lead to poor design when motorists driving faster than 130 km/h speed level.
- By revising side friction factor values to reflect international practices, this research discovered that driver discomfort now started to occur at 143 km/h speed level, a speed level that is 13 km/h higher than the previously found driver discomfort speed level
- Also, this research determined that whereas with 0% superelevation driver discomfort would occur when driving faster than 152 km/h, with proper superelvation design driver discomfort starts to occur only when driving faster than 160 km/h. This implies that proper superelevation application is such an important design element that engineers should pay extra attention in its application particularly in higher speed freeway design.

## REFERENCE


AASHTO(2004), **A Policy on Geometric Design of Highways and Streets-5th edition**, American Association of State Highway and Transportation Officials, America, Washington D.C.

Emmerson, J (1969), Speeds of Cars on Sharp Horizontal Curves, **Traffic Engineering and Control**

K. Fitzpatrick, K. Zimmerman, R. Bligh, S. Chrysler, and Byron Blaschke (2007), **Criteria For High Design Speed Facilities**, Texas Transportation Institute, America

Lamm R,  Basil P., Theodor M.(1999), **Highway Design and Traffic Safety Engineering Handbook**, McGraw-Hill Professional Publishing

Lamm R , Choureiri, E , Mailaender, T(1991), Side friction Demand Versus Side Friction Assumed for Curve Design on Two-Lane Rural Highway, **Transportation research record, No. 1303**, 11-21

Lamm R, A.K. Guenther, E.M. Choueiri(1995), Safety Module for Highway Geometric Design, **Transportation Research Record, No. 1512**, 7-15

McLean, J. R ,(1983), Speeds on Curves : Side Friction Factor Considerations, **Research Report ARR, No. 126**

MOCT (2000), **Highway Geometric Design Standards and Regulation**, Ministry Of Construction and Transportation, South Korea

Peter Spacek (2004), Influence of Speeds on Design Standards in Switzerland, **Transportation Research Record, No. 1881**, 19-26

# CONSTRUCTING THE DENSITY PREDICTIVE MODELS WITHIN THE RAMP JUNCTION AREAS ON URBAN FREEWAY
## -BASED ON THE 1$^{ST}$ URBAN FREEWAY IN BUSAN CITY-

**Mi-Hyeon KWON, Tae-Gon KIM, Yu-Na JUNG**
*Division of Civil and Environmental Engineering*
*Korea Maritime University*
*Busan, Korea*
a28209@nate.com

**ABSTRACT**

Urban freeway is a primary arterial in a big city, which is connected between the urban area and the suburban area or circulated along the circumference of the suburban area. However, most of the urban freeways in Korea do not play their roles in the urban transportation system, because they are experiencing severe traffic congestion with the incoming or outgoing traffic volumes in the ramp junction areas regardless of the rush hours.
Additionally the design criteria of the expressway cannot be applied to the urban freeway for identifying the travel patterns on the urban freeway, because the travel patterns on the urban freeway are different from those on the expressway. So it is very strongly needed to suggest the appropriate density models with the identification of the roadway and traffic characteristics in the ramp junction areas on the urban freeway.
Thus this study is to collect the roadway and traffic characteristic data in the ramp junction areas on the urban freeway, analyze the roadway and traffic characteristic data in the ramp junction areas on the urban freeway, compare the relationship between the road and traffic characteristics in the ramp junction areas on the urban freeway, and finally construct the optimal density predictive models in the ramp junction areas on the urban freeway.
Also, the density predictive models in this study are compared with those suggested on the US highway capacity manual (HCM) and Korea highway capacity manual (KHCM). Based on the roadway and traffic characteristic analyses, and density model construction and verification in the ramp junction areas on the urban freeway, the density models by the KHCM and HCM are found not to fit for predicting the density characteristics on the urban freeway.

## 1. INTRODUCTION

### 1.1 Background

Urban freeway means a high-speed roadway keeping the free flow speed and carrying tremendous traffic flow rapidly except for the rush hours, as one of the primary arterials in the urban transportation system. However, urban freeway is not a high-speed roadway any more in the big city, because most of the vehicles use the urban freeway, and urban freeway is also jammed with the incoming vehicles regardless of the rush hours. What is worse, it is almost impossible to construct the new transportation facilities whenever new travel demand occurs, because a new budget must be made for expanding the new transportation facilities and the new transportation facilities must be coincident with the priority order of the facilities planned. Thus it is absolutely needed to increase the efficiency of the existing urban freeway instead of constructing the new urban freeway.

### 1.2 Objective

Urban freeway, which accommodates almost the free flow speed under the prevailing roadway and traffic conditions except for the rush hours, is composed of mainline segment, weaving segment, ramp and ramp junction like an expressway. However, there is a considerable difference in speed limit, grade, width, and length between urban freeway and expressway due to the land use limited, because urban freeway is located in the urban area, but expressway is located in the rural area. Additionally, there are the clearer congestion and non-congestion sections, the higher travel demand, and the shorter travel length in urban freeway than in the expressway. Until now, a study has been hardly made for urban freeway, despite the marked differences between urban freeway and expressway in Korea. Thus, the purpose in this study is to develop the density predictive model (DPM) based on the traffic and geometric characteristic data in the urban freeway ramp influence areas,

compare the density predictive model developed with the ones in the US highway capacity manual (HCM) and Korean highway capacity manual (KHCM), and finally suggest the improved density predictive model appropriate for the urban freeway ramp influence areas.

### 1.3 Literature Review

According to the US highway capacity manual (HCM, 2000), the level of service (LOS) in ramp influence areas is determined by density for all cases of stable operation, represented by LOS A through E. LOS F exists when the total flow departing from the merge area exceeds the capacity of downstream freeway segment. No density will be predicted for such cases. [1]
According to the Korean highway capacity manual (KHCM, 2005), the LOS criteria follow those in the US highway capacity manual (HCM, 2000), and the LOS criteria for ramp influence areas are shown in Table 1. Especially, density in the ramp influence area is predicted from the converted passenger cars using the traffic flow data collected in the field. [5]

Table 1. LOS criteria for merge and diverge areas

| LOS | Density (pc/km/ln) |
|---|---|
| A | $\leq$6 |
| B | >6-12 |
| C | >12-17 |
| D | >17-22 |
| E | >22 |
| F | Demand exceed capacity |

### 1.4 Data Collection

Specifically, urban freeway under the study is a divided and elevated highway having 2 lanes in each direction, and also has 7 off-ramps, 7 on-ramps, and 5 tunnels. The geometry and equipments were as shown in Figure 1.The speed limits are 80km/h on the mainline section and 50km/h on the ramp section, and the speed surveillance cameras are also installed on the mainline section in order to control the speedup of the vehicles (see Table 2). So, data collection was conducted in the ramp influence areas (Munhyeon (A), Daeyeon (B), Mangmi (C), and Wondong (D)) selected for the analyses of the roadway and traffic characteristics during April through June, 2008. And a master dataset was generated every 15 minutes by the detectors (NC-97) which were installed at the upstream, downstream and ramp locations in the merge and diverge influence area of urban freeway. It was converted into a data format for visual and statistical inspection via a spread sheet, and used for analyses. Occasionally the detectors produced bad data, and only valid data for all 24 hours were used in the analyses.

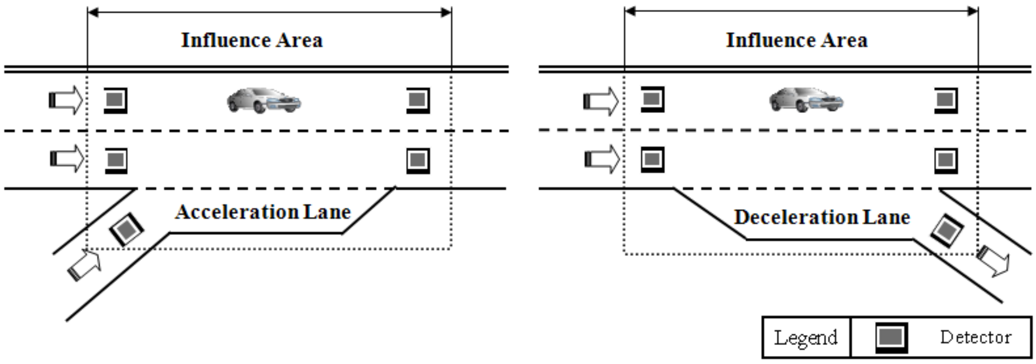


Figure 1. Urban freeway under the study, ramp geometries and detector locations

Table 2. Geometric characteristics in the merge and diverge influence areas

| Ramp junction | A | B | C | D |
|---|---|---|---|---|
| Length of deceleration (m) | 80 | 170 | 150 | 120 |
| Length of acceleration (m) | 130 | 230 | 120 | 230 |
| No. of lanes (Mainline /Ramp) | 2 / 1 | | | |
| Lane width (m) (Mainline /Ramp) | 3.7 / 3.7~5.0 | | | |
| Speed limits (km/h), (Mainline /Ramp) | 80 / 50 | | | |
| Free flow speed (km/h), (Mainline /Ramp) | 60 / 50 | | | |

## 2. Analysis of Traffic Characteristics

For the microscopic analyses of the traffic characteristics, flow was converted into the flow rate (pc/h) using the heavy vehicle factor, speed was converted into the space mean speed by the speed data of the detectors installed on the merge and diverge influence area, and density was also estimated by the reciprocal of the headway distance, which was the headway time multiplied by the space mean speed in the above.

### 2.1 Flow Rate

Flow rate was expressed by a vehicle per 15 minutes (pc/15min.), as a number of vehicles which passed the detector (NC-97) for a unit period, and converted into the hourly volume as shown in the below.

$$V_t = \frac{N}{T \times f_{HV} \times PHF}$$

$$V = \sum_{i=1}^{4} V_i$$

Where,

$N$ : No. of vehicles passed the detector

$T$ : Unit period (15 minutes)

$f_{HV}$ : Adjustment factor for heavy vehicle $(1/[1 + P_{T1}(E_{T1} - 1) + P_{T2}(E_{T2} - 1)])$

$PHF$ : Peak hour factor

$V$ : Hourly volume (pc/h)

$V_i$ : Volume for 15 minutes (pc/15 min.)

The average flow rates appeared to be approximately distributed within about 600pc/h to 1,000pc/h in the ramp influence areas. And the minimum and maximum flow rates (capacities) were expected to be within about 70pc/h to 170pc/h and about 1,100pc/h to 1,600pc/h in the ramp influence areas, respectively. Also, the flow rates in the daytime period from 07:00 to 20:00 appeared to be

approximately distributed within about 700pc/h to 1,400pc/h in the ramp influence area A, about 1,000pc/h to 1,400pc/h and about 700pc/h to 1,600pc/h in the ramp influence areas B and C, and about 700pc/h to 1,200pc/h in the ramp influence area D as shown in Table 3 and Figure 2. So, the flow rate distribution proved to be very effective in identifying the existence or nonexistence of a typical pattern at the peak periods in the ramp influence areas.

Table 3. Results of flow rate characteristic analysis

| | | Flow rate (pc/h) | | | |
|---|---|---|---|---|---|
| | | Min | Max | Average | 07:00～20:00 |
| On-Ramp | A | 112 | 1,354 | 668 | 1,200～1,400 |
| | B | 142 | 1,408 | 966 | 1,300～1,400 |
| | C | 142 | 1,562 | 995 | 1,400～1,600 |
| | D | 74 | 1,198 | 629 | 1,000～1,200 |
| Off-Ramp | A | 97 | 1,268 | 746 | 700～1,300 |
| | B | 112 | 1,337 | 880 | 1,000～1,400 |
| | C | 169 | 1,339 | 931 | 700～1,400 |
| | D | 139 | 1,157 | 811 | 700～1,200 |

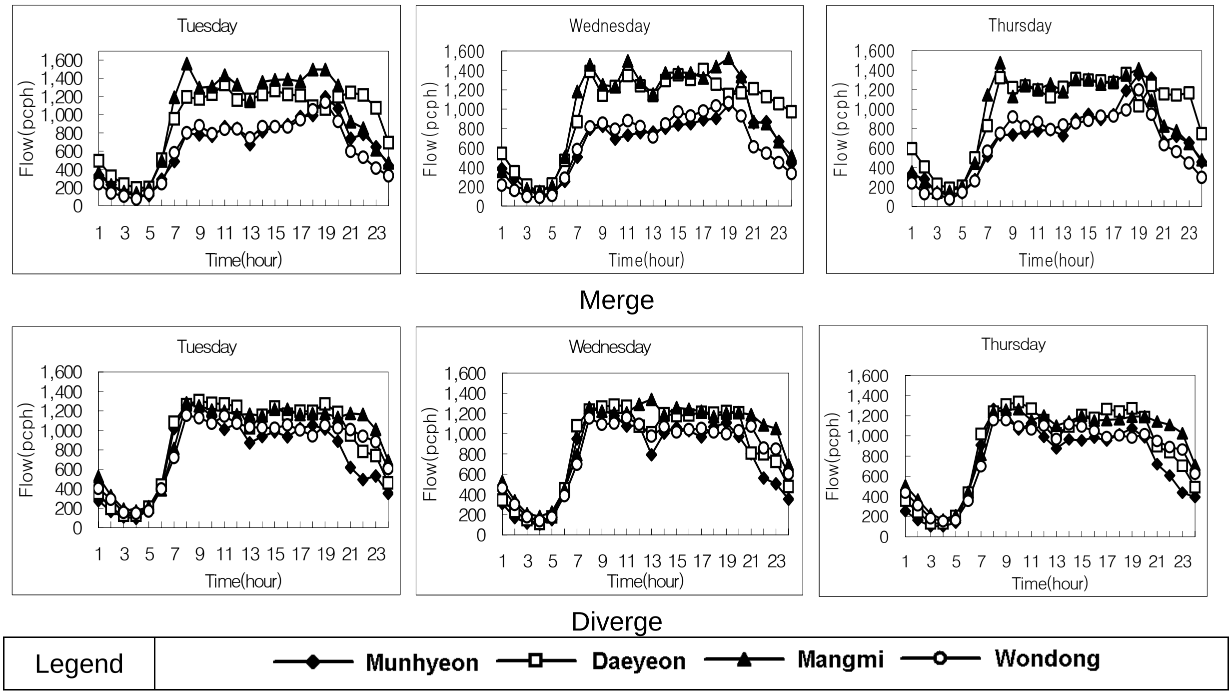



Figure 2. Flow rate distribution

**2.2 Speed**

Speed was expressed by a kilometer per hour (km/h) as the distance which the vehicles traveled for a unit period, and converted into the space mean speed (May, A. D., 1990, [4]) as shown in the below.

$$u_t = \frac{\sum n_i u_i}{\sum n_i}$$

$$u_s = \frac{\sum n_i}{\sum \frac{n_i}{u_i}}$$

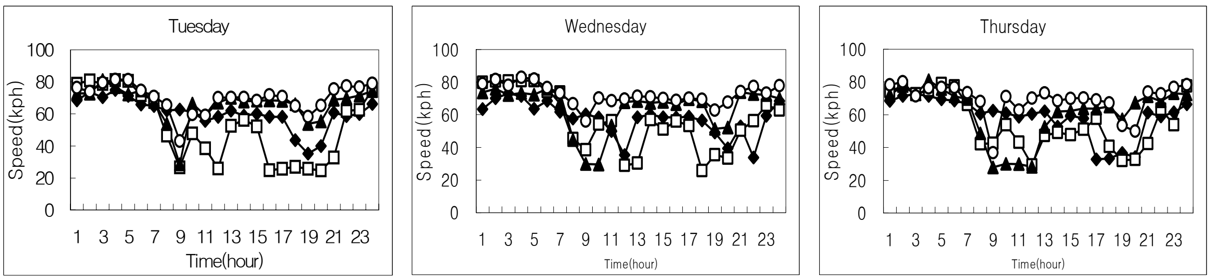
Where,

| | |
|---|---|
| $n_i$ | : No. of vehicle in the speed class (pc/15min) |
| $\sum n_i$ | : No. of vehicle for every 15 minutes (N) |
| $u_i$ | : Mean speed in the speed class (km/h) |
| $u_s$ | : Space mean speed converted (km/h) |
| $u_t$ | : Time mean speed observed (km/h) |

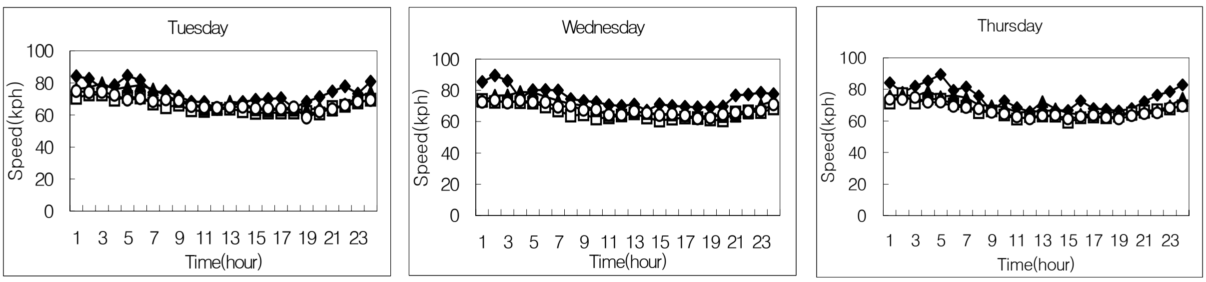
The average speeds appeared to be approximately distributed within about 50km/h to 75km/h in the ramp influence areas. And the minimum and maximum speeds were expected to be within about 20km/h to 65km/h and about 70km/h to 90km/h in the ramp influence areas, respectively. Also, the speeds in the daytime period from 07:00 to 20:00 appeared to be approximately distributed within about 25km/h to 80km/h in the ramp influence area A, about 20km/h to 75km/h in the ramp influence areas B, C, and D as shown in Table 4 and Figure 3. So, the speed distribution proved to be very effective in identifying the existence or nonexistence of delay at the peak periods in the ramp i nfluence areas.

Table 4. Results of speed characteristic analysis

| | | Speed (km/h) | | | |
|---|---|---|---|---|---|
| | | Min | Max | Average | 07:00~20:00 |
| On-Ramp | A | 32.8 | 75.0 | 59.0 | 25~35 |
| | B | 24.6 | 81.1 | 54.6 | 20~30 |
| | C | 27.9 | 81.1 | 63.8 | 20~30 |
| | D | 36.9 | 82.9 | 70.7 | 20~30 |
| Off-Ramp | A | 63.7 | 89.5 | 74.3 | 60~80 |
| | B | 59.2 | 77.3 | 66.0 | 55~70 |
| | C | 62.3 | 79.8 | 69.5 | 60~75 |
| | D | 58.2 | 74.9 | 67.1 | 55~70 |




Merge



Diverge

| Legend | Munhyeon, Daeyeon, Mangmi, Wondong |
|---|---|

Figure 3. Speed distribution

**2.3 Density**

Density was expressed by a vehicle per kilometer (pc/km) as the number of vehicles which was traveling in the unit length of roadway, and estimated by the reciprocal of the headway distances (TRB, 1975, [2]) as shown in the below.

$$h = \frac{\sum (t_i - t_{n-1})}{\sum n_i}$$

$$k = \frac{3.6}{h_i \times u_{si}}$$

$$k_i = \frac{k_{U1} + k_{D1}}{2} \;,\; k_2 = \frac{k_{U2} + k_{D2}}{2}$$

$$D = \frac{k_1 + k_2 + k_3}{3}$$

Where,

| | |
|---|---|
| $h$ | : Mean headway for 15 min. (sec) |
| $h_i$ | : Headway for every 15 min. (sec) |
| $t_i$ | : Arrival time of vehicle $i$ (sec) |
| $t_{i-1}$ | : Arrival time of vehicle $i-1$ (sec) |
| $u_{si}$ | : Space mean speed for every 15 min (km/h) |
| $k_1$ | : Density of 1st lane from adjacent lane for 15 min (pc/15 min.) |
| $k_2$ | : Density of 2nd lane from adjacent lane for 15 min (pc/15 min.) |
| $k_3$ | : Density of adjacent lane for 15 min (pc/15 min.) |
| $k_{U1}$ | : Density of upstream 1st lane for 15 min (pc/min.) |
| $k_{U2}$ | : Density of upstream 2nd lane for 15 min (pc/min.) |
| $k_{D1}$ | : Density of downstream 1st lane for 15 min (pc/min.) |
| $k_{D2}$ | : Density of downstream 2nd lane for 15 min (pc/min.) |
| $D$ | : Mean density in the merge and diverge influence area (pc/15 min.) |

The average densities appeared to be approximately distributed within about 10pc/km to 25pc/km in the ramp influence areas. The minimum densities were expected to be within about 1pc/km to 2pc/km in the ramp influence areas. The maximum densities (jam densities) were also expected to be distributed within about 35pc/km to 55pc/km in the merge influence areas, and about 20pc/km to 30pc/km in the diverge influence areas. The densities from 07:00 to 09:00 appeared to be approximately distributed within about 30pc/km to 60pc/km in the merge influence area, about 10pc/km to 30pc/km in the diverge influence area, as shown in Table 5 and Figure 4. So, the density distribution proved to be very effective in identifying the existence or nonexistence of a jam at the peak periods in the ramp influence areas.

Table 5. Results of density characteristic analysis

| | | Density(pc/km) | | | |
|---|---|---|---|---|---|
| | | Min | Max | Average | 07:00~20:00 |
| On-Ramp | A | 2 | 38 | 17 | 30~40 |
| | B | 2 | 51 | 25 | 50~60 |
| | C | 2 | 54 | 18 | 50~60 |

| | | | | | |
|---|---|---|---|---|---|
| | D | 1 | 35 | 13 | 30~40 |
| Off-Ramp | A | 1 | 20 | 11 | 10~20 |
| | B | 2 | 22 | 14 | 10~25 |
| | C | 2 | 27 | 16 | 10~30 |
| | D | 2 | 30 | 14 | 10~30 |

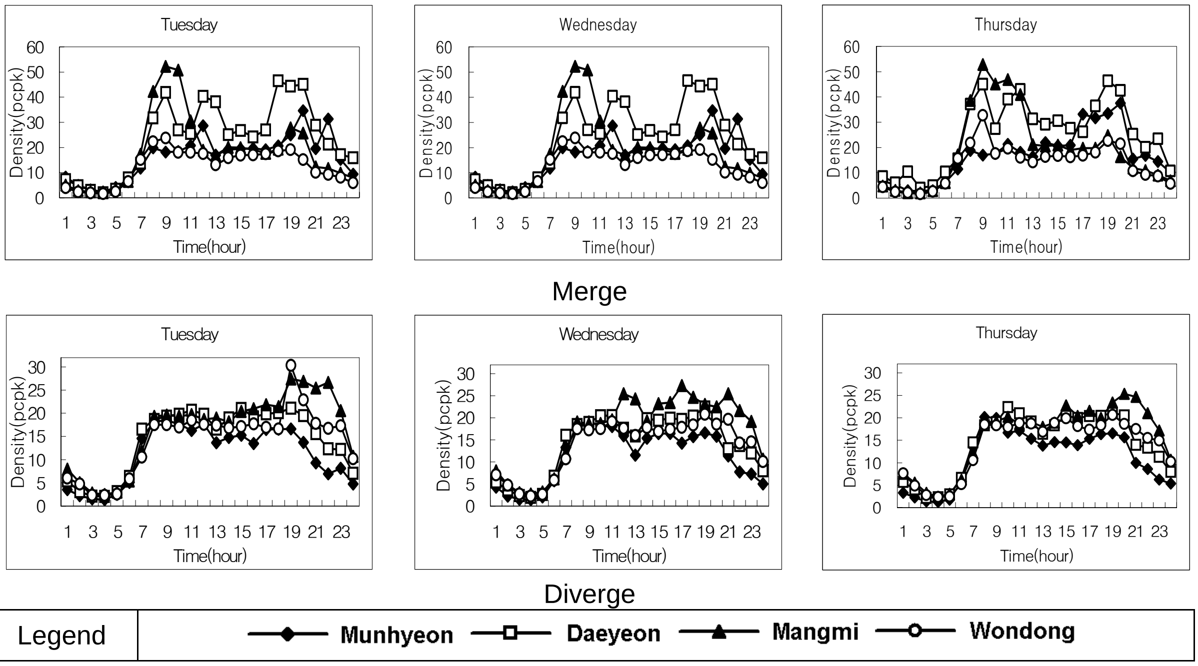


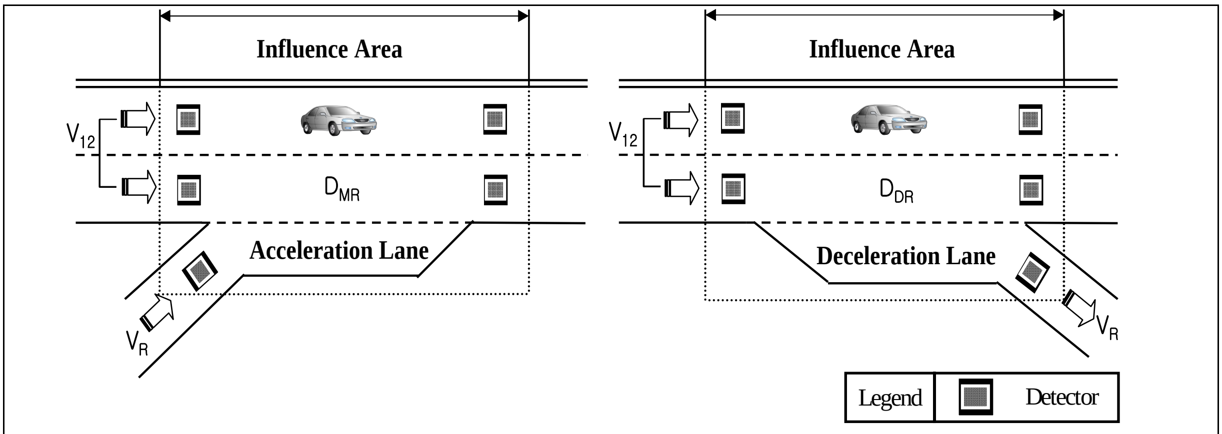
Figure 4. Speed distribution

## 3. Model development and Validation

The density predictive model was developed with a portion of traffic data collected in the ramp influence areas by using the multiple regression analysis, and validated by using the correlation analysis as well.

### 3.1 Model Development

In the density predictive model, ramp flow, flow of lane 1 and 2 from the adjacent lane, and length of change speed lane were used as the dependent variables, mean density in the ramp influence area as the independent one as shown in Figure 5;




| Independent variables | |
|---|---|
| $V_R$ | : Ramp flow (pc/h) |

| | |
|---|---|
| $V_{12}$ | : Flow of lane 1 and 2 from adjacent lane (pc/h) |
| $L_A$ | : Length of acceleration lane (m) |
| $L_D$ | : Length of deceleration lane (m) |
| Dependent variables | |
| $D_{MR}$ | : Mean density in the merge influence area (pc/km/l) |
| $D_{DR}$ | : Mean density in the diverge influence area (pc/km/l) |

Figure 5. Sketch of ramp junctions and definition of the variables

Under the assumption that the density in the ramp influence area would be influenced by the traffic and geometric characteristics in the ramp influence area, especially, ramp flow, flow of lane 1 and 2 from adjacent lane, length of acceleration and deceleration lanes, the density predictive model ( $f(k)$ ) was suggested as follows;

$f(k) = \beta_0 + \beta_1 V_{12} + \beta_2 V_R + \beta_3 L_A$ : Merge influence area

$f(k) = \beta_0 + \beta_1 V_{12} + \beta_2 L_D$ : Diverge influence area

Where,

$\beta_0, \ldots, \beta_3$ : Regression coefficients

The multiple regression analysis was used to build the density predictive model, which were developed by the all-possible-regression selection procedures for the purpose of identifying the important independent variables with the criteria of $R^2$. Particularly, the multi-collinearity was also avoided by the trial-and-error process.

Table 6. Result of density predictive modeling

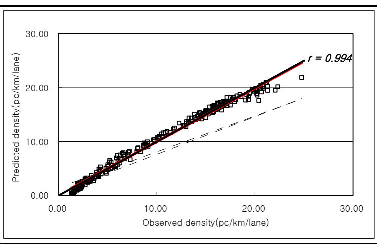
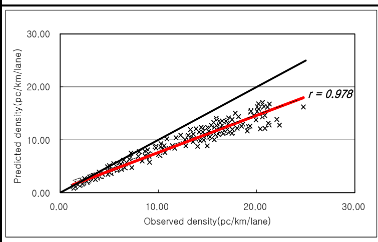
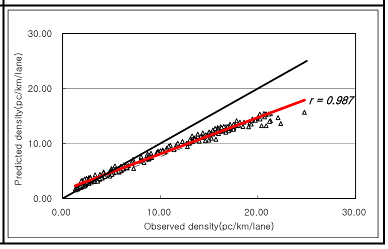
| Model | $R^2$ | $\Pr ob > F_{1)}$ |
|---|---|---|
| $D_{MR} = 1.02644 + 0.005899V_{12} + 0.009646V_R - 0.010058L_A$ | 0.986 | 0.000 |
| $D_{DR} = 0.94158 + 0.006352V_{12} - 0.007434L_D$ | 0.931 | 0.000 |

Annotation 1) $(P > |t|) = (P - value)$

So, the density predictive model proved to be determined by ramp flow, flow of lane 1 and 2 from the adjacent lane, and length of the acceleration lane as shown in Table 6.

**3.2 Model Validation**

There were two approaches applied to ensure the validity of the models developed. One approach was to conduct t-test between the observed and expected densities, whether the p-values were greater than the significance level ( $\alpha / 2 = 0.025$) or not at the 95% confidence level as shown in Table 7. They were fully less than the p-values in the ramp influence areas under the study. Another was to test the utility of the regression models with traffic data unused. The results ( $r$ ) of the correlation analysis were shown to be 0.968 and 0.994 in the ramp influence areas regardless of the urban and suburban areas as shown in Figure 6. So, these models proved to be very effective in predicting densities in the ramp influence areas on the urban area.

| | This model | KHCM model | HCM model |
|---|---|---|---|
| On-Ramp |  |  |  |

Off-Ramp

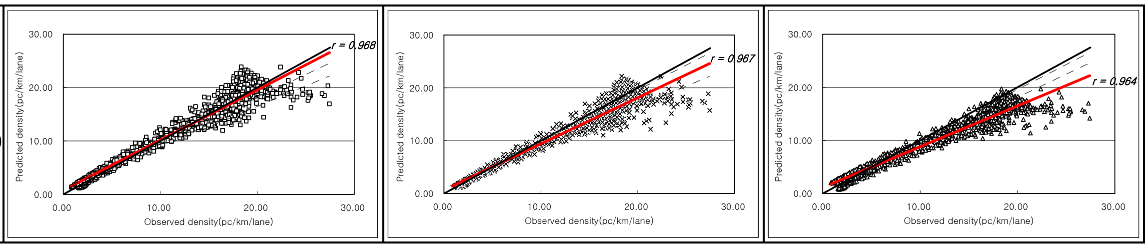



Figure 6. Correlations between the observed and expected densities

Table 7. Results of t-tests between the observed and expected densities

| | | t-value | p-value | Result |
|---|---|---|---|---|
| On-Ramp | KHCM | 17.65133 | 0.000 | Reject |
| | HCM | 12.62977 | 0.000 | Reject |
| | This Model | -0.75129 | 0453 | Accept |
| Off-Ramp | KHCM | 17.538 | 0.000 | Reject |
| | HCM | 28.474 | 0.000 | Reject |
| | This Model | -0.004 | 0.997 | Accept |

## 4. Model Evaluation

The statistics were applied to evaluate the measures of effectiveness (MOE) between this model, the HCM model, and the KHCM model. The statistics applied were to compare the root mean square error (RMSE) between the observed and expected densities by the models.

Particularly, there was the more difference in root mean square error (RMSE) of this model (0.981~1.768) than those in the KHCM model (2.056~3.205) and the HCM model (2.813~2.861), respectively as shown in Table 8. So this model proved to have a higher predictability than the KHCM and HCM models in urban freeway ramp influence area.

Table 8. RMSE results of density models predicted

| | This Model | KHCM Model | HCM Model |
|---|---|---|---|
| On-Ramp | 0.981 | 3.205 | 2.861 |
| Off-Ramp | 1.768 | 2.056 | 2.813 |

## 5. Conclusions

From the traffic characteristic analyses, and the development and validation of the predictive density model in the ramp influence areas, the following conclusions were drawn;

ⅰ) Traffic characteristics distributed were shown to be very effective in identifying their distinct characteristics at the peak periods in the urban freeway ramp influence areas,

ⅱ) Daytime period from 7:00 to 20:00 has a higher density than the nighttime period regardless of the location or direction.

ⅲ) Density in the ramp influence area was shown to be highly correlated by the ramp flow ($V_R$), flow of lane 1 and 2 from adjacent lane ($V_{12}$), and length of speed change lane ($L_A, L_D$) and,

ⅳ Density predictive model were shown to be higher in the explanatory power ($R^2$) than the KHCM and HCM models with about 0.931~0.986 in the ramp influence areas under the study.

It was concluded that this study was needed to be continued under the various geometric characteristics for the purpose of the reliability of the density predictive model.

## References

1. America Association of State Highway and Transportation Officials, A Policy on Geometric Design of Highways and Streets, AASHTO, Washington, D.C., 2004

2. Bowerman, B.L., and O' Connell, R.T., Linear Statistical Model; an applied approach , PWS-KENT Publishing Company(1990)

3. Gartner, N., Messer, C.J., and Rathi, A.K., Traffic Flow Theory; A Monograph, Special Report 165, Revised Edition, Transportation Research Board, National Research Council, Washington, D.C., 1997

4. Korea Ministry of Construction and Transportation, Korea Highway Capacity Manual, KHCM, Korea, 2005

5. Transportation Research Board, Highway Capacity Manual, National Research Council Washington, D.C., 2000

# FACTORS INFLUENCING FREIGHT TRUCK ROUTE SELECTION

Freight Characteristics Influencing the Ratio of Freight Truck Expressway Use

**Hirotaka SEKIYA, Masanori KOBAYASHI, Hiroyuki NAMBU, Katsumi UESAKA**

*Traffic Engineering Division, National Institute for Land and Infrastructure Management, Ministry of Land, Infrastructure, Transport and Tourism, Japan*

*sekiya-h92tb@nilim.go.jp*

## ABSTRACT

This article details how freight characteristics influence freight truck route selection. Specifically, relationships between the ratio of freight truck expressway use and four factors that describe freight characteristics were analyzed; the four factors are transport distance, delivery time specification, facility type from which freight is shipped, and freight volume. Based on the results of this analysis, we developed a generalized logit model that predicts the ratio of expressway use from the above factors, helping us understand how the trend toward more sophisticated freight transport will influence freight truck route selection in the future.

## 1. INTRODUCTION

Route selection models for cars are often developed with a particular emphasis on traffic conditions determined by the geometric factors of roads, such as congestion speed and maximum traffic volume. However, freight trucks must select routes taking into account not only traffic conditions but the characteristics of the freight that they are transporting. For example, trucks are more likely to choose expressways if the time of delivery is specified, as doing so enables punctual arrival at their destination; in contrast, general roads often cause large delay in delivery due to unexpected congestion. If a truck is carrying perishables, or other goods that must be kept chilled, it is more likely to take expressways to shorten travel time.

This article discusses the relationships between freight characteristics and freight truck route selection in which the ratio of expressway use was used as an index of freight truck route selection.

## 2. SUMMARY OF THE DATA USED

This study employs data obtained from the Commodity Flow Census, which has been conducted every five years since 1970 to investigate actual freight movement throughout the country. The census surveys such items as those shown in Table 1 regarding individual freight shipped from subject offices during three specific days in the census year.   An Outline of the census and the data obtained from it are shown in Table 1 and Table 2. Expressway networks in Japan are shown in Figure 1.

The analysis was conducted using 322,662 records of freight transported by trucks between prefectures; these records appear at the bottom in Table 2. More than half of the records lack information on whether or not the freight was transported via expressway, as the survey was conducted for consignors, who are unlikely to know the actual transport routes, rather than for forwarders.

Table 1 Outline of the Commodity Flow Census

| | |
|---|---|
| **Outline of the census** | |
| Date of survey | 18-20 October 2005 ( three consecutive weekdays) |
| Subjects surveyed | Consignors of 21,763 offices in four industries: mining, manufacture, wholesale, and warehouse<br>(This accounts for 3.2% of the total number of offices in the four industries) |
| Freight surveyed | Raw materials, products, and merchandise that are transported from and to the subjects surveyed (shown above) |
| **Items surveyed (examples)** | |
| Detailed freight types | 85 types: fruit, precision instruments, clothing, etc.<br>(these 85 types are categorized into nine freight types) |
| Primary means of transportation | 12 categories: trucks(5), railways(2), ships(3), airplanes |
| Relay facility | Fill in the name of the railway station, port, airport, or wholesale market |
| OD of freight transport | Fill in the address at the municipal level |
| Expressway use | Used or not-used |
| Interchanges | Fill in the name of interchanges used |
| Delivery time specification | Hour specified, AM/PM specified, date specified, no specification |
| Facility type<br>from which freight is shipped | Refrigerator warehouse, open warehouse, etc. |

Table 2 Outline of data used (number of records)

| | | | |
|---|---|---|---|
| Total number of records | 1,126,545 | *100.0%* | |
| Export | 11,883 | *1.1%* | |
| Domestic | 1,114,662 | *98.9%* | *100.0%* |
| Primary means of transportation: ship or railway | 5,473 | | *0.5%* |
| Primary means of transportation: truck | 1,109,189 | *100.0%* | *99.5%* |
| Transported within the prefecture | 329,997 | *29.8%* | |
| Transported between prefectures | 735,226 | *66.3%* | *100.0%* |
| Unclear whether or not expressways were used | 412,564 | | *56.1%* |
| Clear whether or not expressways were used | **322,662** | | *43.9%* |

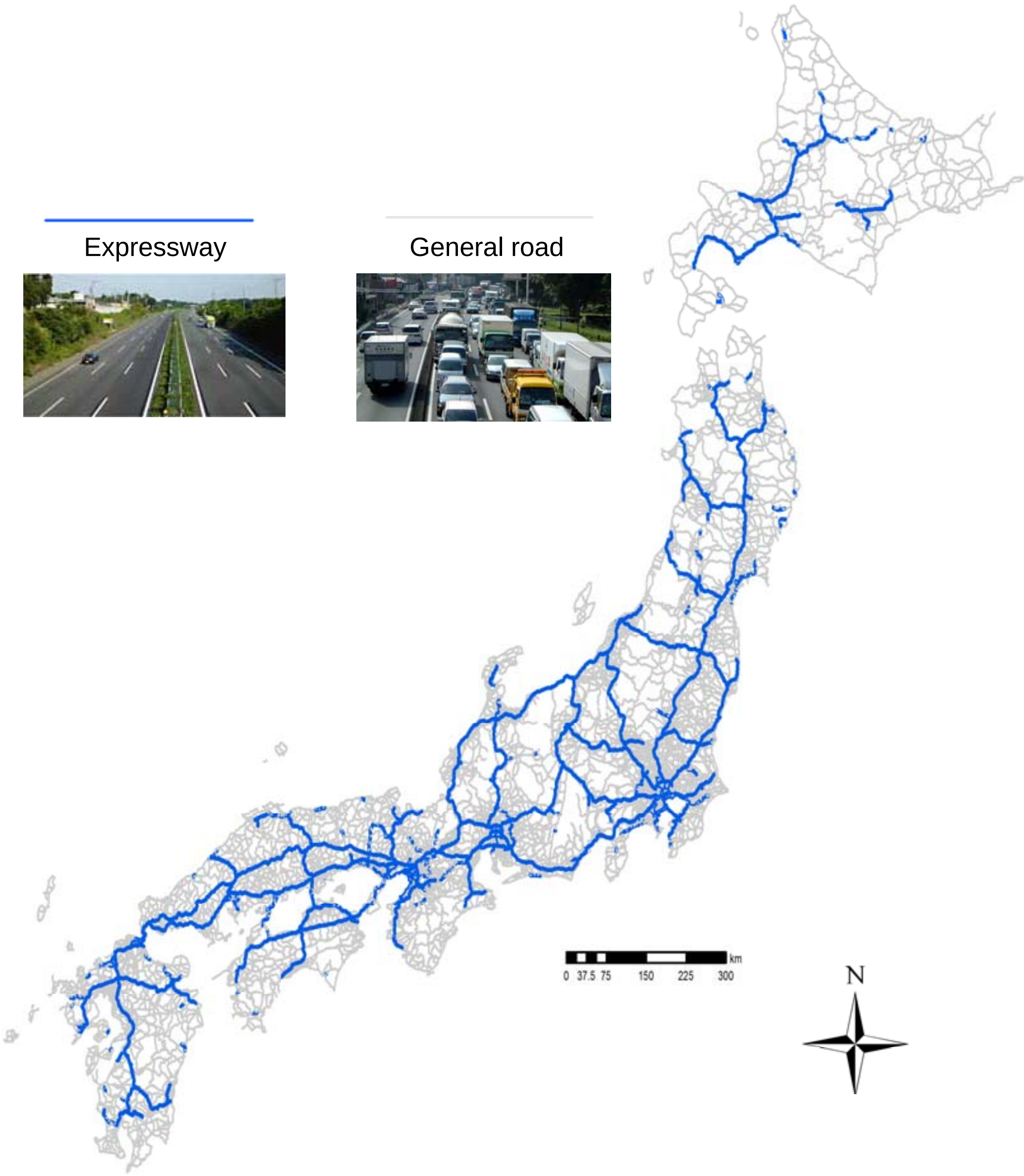



Figure 1 Expressway networks in Japan

## 3. RELATIONSHIPS BETWEEN THE RATIO OF EXPRESSWAY USE AND FREIGHT CHARACTERISTICS

Figure 2 shows average ratios of expressway use by 100 kilometer distance category for nine freight types, indicating that, for the most part, the ratio of expressway use increases until 500 km and then becomes flat for all nine freight types. Clear differences in the ratio of expressway use were observed among these types; the ratio in Agriculture and Fishery is higher, while the ratio in Mining is especially low. The ratio of expressway use in Forestry and Mining are shown for only distance categories of 500km and below, as the census was unable to obtain a sufficient number of records for distance categories of above 500km.

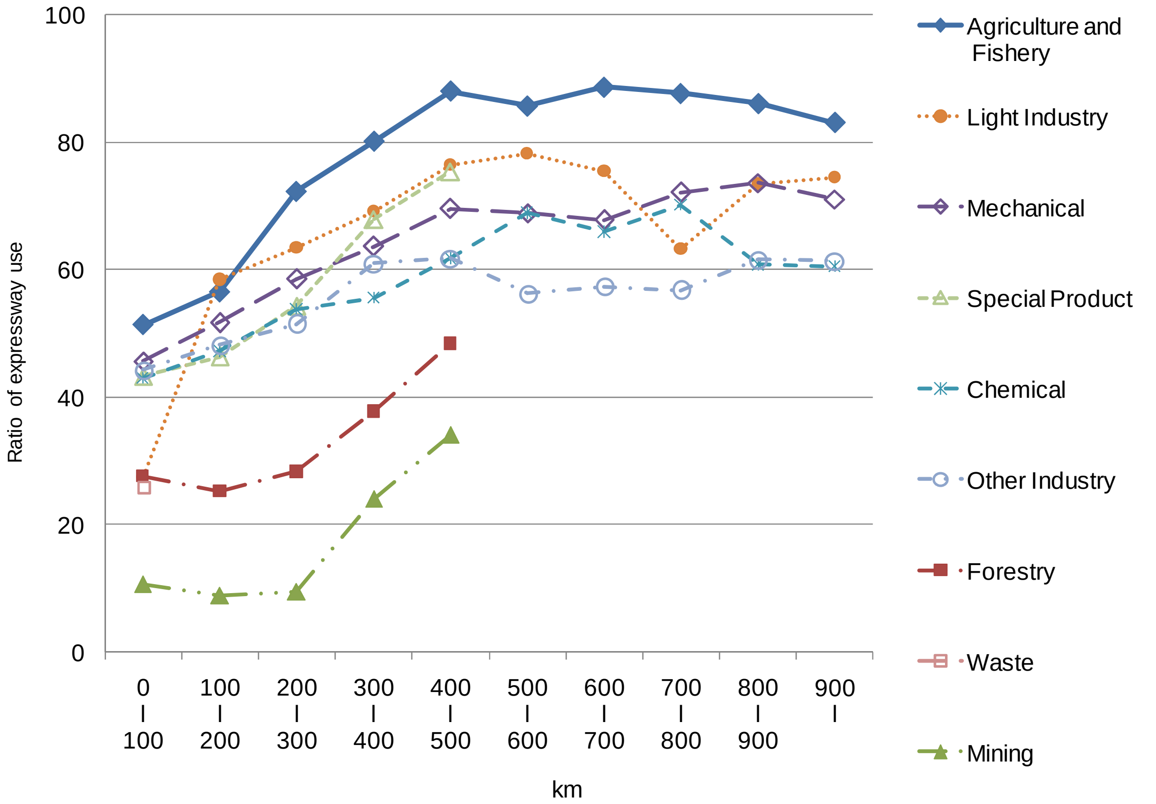



Figure 2 Average ratios of expressway use (for nine freight types)

### 1) Freight Shipped from Refrigerator Warehouses

The ratio of expressway use in the Agriculture and Fishery industry is the highest among the nine freight types over all distance categories except 100-200km. This can be attributed to the hypothesis that the ratio of expressway use of perishables such as raw fish, for which express transport is preferred in order to preserve freshness, is high, pushing up the overall ratio of expressway use in Agriculture and Fishery, which partly consists of perishables. In particular, an even higher ratio of expressway use is expected for transport by trucks equipped with a refrigerator or freezer because of a greater incentive to shorten transport time in an attempt to conserve energy during transport.

In order to test this hypothesis, the ratio of expressway use for freight shipped from refrigerator warehouses was compared to that of freight shipped from other facilities. As shown in Figure 3, the former showed a higher ratio of expressway use than the latter. As shown in Table 3, the results of a Z-test using Equation 1 tells us statistically significant evidence of differences in the ratios of expressway use at a 0.1% confidence level; therefore, trucks carrying freight shipped from refrigerator warehouses are more likely to use expressways.


Figure 3 Ratio of expressway use
(Freight shipped from refrigerator warehouses v.s. Freight shipped from other facilities)

$$Z = \frac{P_a - P_b}{\sqrt{P_{average} \times (1 - P_{average}) \times \left(\frac{1}{N_a} + \frac{1}{N_b}\right)}} \quad (1)$$

$P_a$: Ratio of expressway use for freight shipped from refrigerator warehouses
$P_b$: Ratio of expressway use for freight shipped from other facilities
$P_{average}$: Average of Pa and Pb weighted by the number of samples
$N_a$: the number of samples of freight shipped from refrigerator warehouses
$N_b$: the number of samples of freight shipped from other facilities

Table 3 Result of significance test

| km | Ratio of expressway use | | | Number of sample | | Z value | |
|---|---|---|---|---|---|---|---|
| | $P_a$ | $P_b$ | Pave | $N_a$ | $N_b$ | | |
| 0-100 | 49.8% | 38.7% | 39.0% | 2,736 | 76,761 | 11.76 | *** |
| 100-200 | 65.4% | 50.2% | 50.6% | 1,688 | 74,965 | 12.34 | *** |
| 200-300 | 76.3% | 56.7% | 57.0% | 646 | 43,171 | 9.99 | *** |
| 300-400 | 77.6% | 61.9% | 62.2% | 505 | 34,898 | 7.21 | *** |
| 400-500 | 85.8% | 67.0% | 67.3% | 494 | 34,606 | 8.86 | *** |
| 500- | 73.5% | 68.6% | 68.7% | 741 | 51,451 | 2.87 | ** |
| Total | 63.5% | 54.4% | 54.6% | 6,810 | 315,852 | 14.79 | *** |

Signif. codes: 0 ‘***’ 0.001 ‘**’ 0.01 ‘*’ 0.05 ‘.’ 0.1 ‘ ’ 1

### 2) Freight with Delivery Time Specified

More sophisticated freight services such as just-in-time transport and time-specified delivery have been advancing, enabling freight to be transported so as to arrive at a time adjusted to a factory's manufacturing processes, or during lunchtime or evening shopping hours. Figure 4 shows the percentage of freight in terms of delivery time specification: (1) hour specified, (2) AM or PM specified, (3) date specified, and (4) not-specified. The percent of freight with the hour specified has increased by 1.5 points during the ten years between 1995 and 2005. Agriculture and Fishery shows the highest percentage of hour specified freight among the nine freight types, with 19.2% of shipments designated as such.

We established a hypothesis that hour specified freight is more likely to be transported via expressway, which are known to be more travel time reliable, as such freight is not allowed any delay in delivery. The hypothesis was tested by comparing the ratio of expressway use of hour specified freight and that of not-specified freight in Agriculture and Fishery. As shown in Figure 5, hour specified freight indicated greater ratios of expressway use over all distance categories except 400-500km. As shown in Table 4, a Z-test revealed that the ratio of expressway use for hour specified freight was statistically higher than that of not-specified freight at a significance level of 0.1% over all distance categories except 300-400km and 400-500km. This result suggests that expressways are used at a greater rate when hour specified freight is transported.

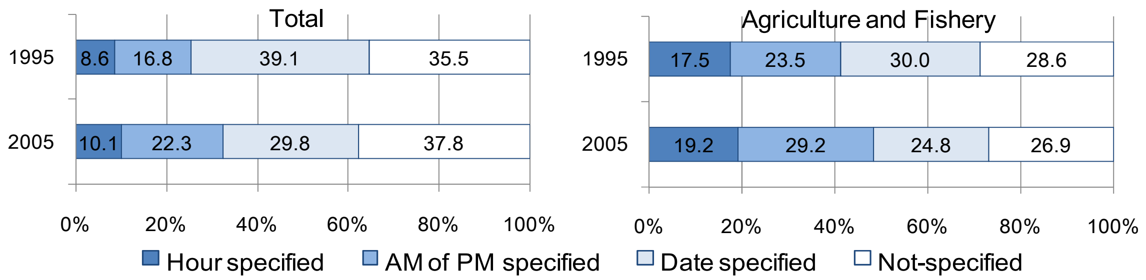



Figure 4 Percentage of freight in terms of delivery time specification
(Left: Sum of the nine freight types, Right: Agriculture and Fishery)

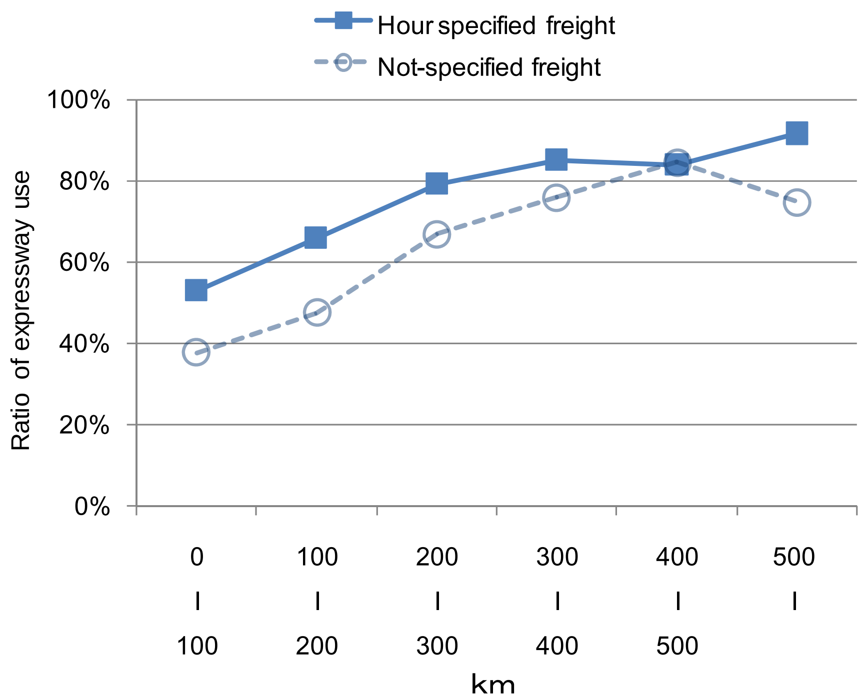



Figure 5 Ratio of expressway use
(Hour specified freight v.s. Not-specified freight)

Table 4 Result of significance test

| km | Ratio of expressway use $P_a$ | $P_b$ | $P_{ave}$ | Number of sample $N_a$ | $N_b$ | Z value | |
|---|---|---|---|---|---|---|---|
| 0-100 | 53.1% | 37.9% | 47.6% | 911 | 515 | 5.54 | *** |
| 100-200 | 66.0% | 47.6% | 60.0% | 911 | 435 | 6.44 | *** |
| 200-300 | 79.3% | 66.9% | 74.7% | 454 | 272 | 3.71 | *** |
| 300-400 | 85.1% | 75.9% | 81.3% | 202 | 141 | 2.17 | * |
| 400-500 | 84.1% | 84.6% | 84.3% | 113 | 117 | -0.11 | |
| 500- | 91.6% | 74.7% | 87.8% | 597 | 174 | 6.00 | *** |
| Total | 70.9% | 55.6% | 65.7% | 3,188 | 1,654 | 10.59 | *** |

Signif. codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

a: Hour specifed freight, b: Not-specified freight

**3) Small-Lot Freight**

Frequency of freight delivery is increasing and the weight of freight shipped simultaneously (freight lot) is decreasing to meet the needs of consignees who prefer that only the necessary amount of freight should be delivered at the most appropriate times in order to reduce unnecessary storage. As shown in Figure 6, average freight lot has halved during the 15 years between 1990 and 2005. Freight lot here is defined as the weight of freight that is shipped as the same kind, at the same time, by the same means of transportation, and to the same destination.

The relationship between freight lot and the ratio of expressway use was analyzed to discuss how the trend of decreasing freight lot influences freight truck route selection. As shown in Figure 7, the smaller the freight lot, the higher the ratio of expressway use; the cause of this trend was inferred from the statistical data shown in Figure 8 and Figure 9. Figure 8 shows the percent of truck types used for freight transport by freight lot category; a: consignors' own trucks, b: forwarders' trucks carrying different items assigned from multiple consignors, c: forwarders' trucks carrying a single item assigned from a single consignor, d: other. The figure shows that the smaller the freight lot, the higher the percent of forwarders' trucks carrying different items (b). Figure 9 shows the average ratio of expressway use for the four truck types; that of forwarders' trucks carrying different items (b) is the highest among them at 62%. Thus, the reason why the ratio of expressway use for small-lot freight is higher is because small-lot freight tends to be transported by forwarders' trucks carrying different items (b), for which we have good evidence of a higher ratio of expressway use.

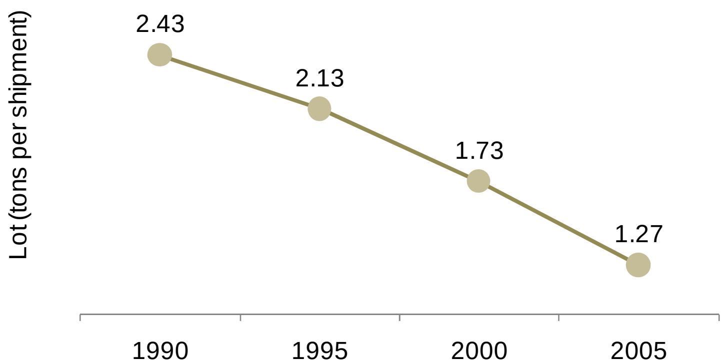



Figure 6 Average freight lot (tons per shipment)

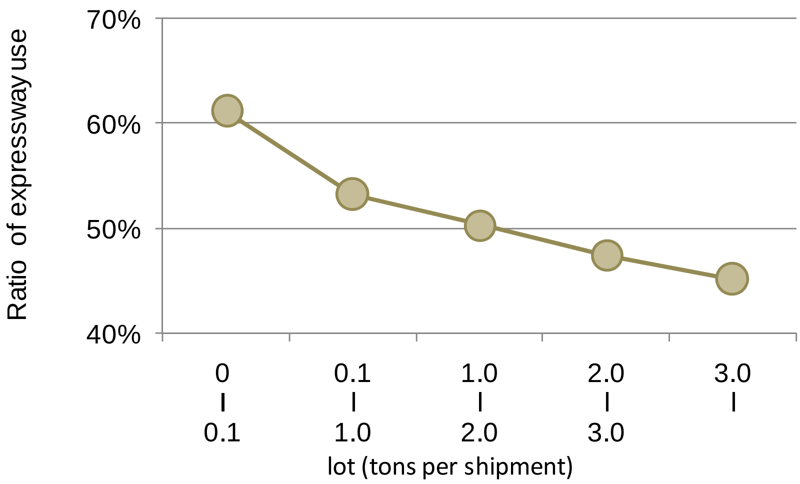
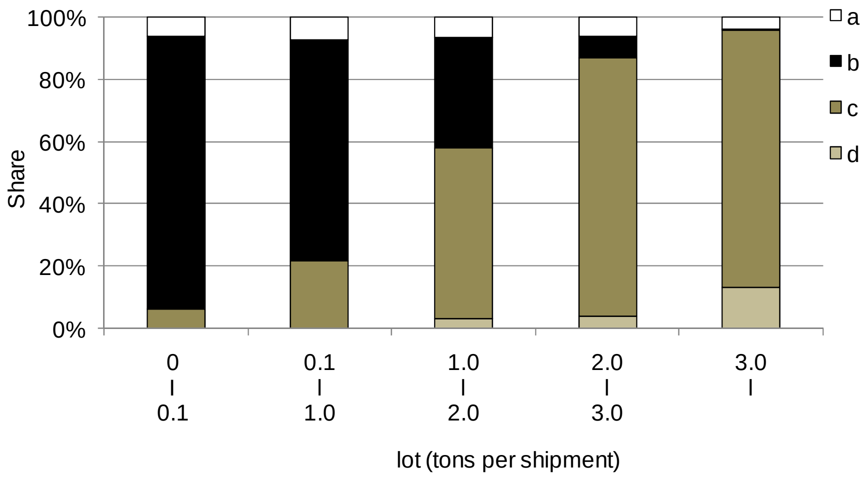


Figure 7 Relationship between freight lot and ratio of expressway use




*a: consignors' own trucks*
*b: forwarders' trucks carrying different items assigned from multiple consignors*
*c: forwarders' trucks carrying a single item assigned from a single consignor*
*d: other*

Figure 8 Percent of truck types used for transport by freight lot

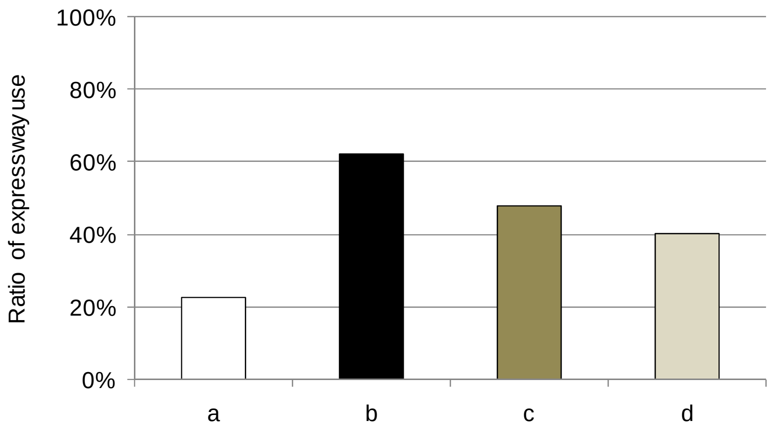


Figure 9 Average ratio of expressway use for the four truck types

## 4. MODEL PREDICTING THE RATIO OF EXPRESSWAY USE

We developed a model that predicts the ratio of expressway use from the freight characteristics discussed previously. Specifically, the generalized logit model shown in Equation 2 was estimated by regressing the ratio of expressway use to four explanatory variables: (1) transport distance, (2) facility type from which freight is shipped, (3) delivery time specification, and (4) freight lot. Out of 12,127 records in Agriculture and Fishery, the 9,353 records which hold data on all four variables were used for the estimation of the model.

Table 5 shows the estimated regression coefficients of the variables; their signs all agreed with the conditions discussed in the previous chapter, and they all satisfied a 0.1% confidence level. This verifies that all four variables are valid factors that explain the behavior of freight truck route selection.

Plots of the figures predicted by the model are shown in Figure 10. Averaged figures are shown in Figure 11 and Table 6; the maximum difference was 5.6 points at 100-200km, and the accuracy ratio of the model was 0.72.

$$Logit(f) = log\left(\frac{f}{1-f}\right) = \alpha_0 + \alpha_1 x_1 + \alpha_2 x_2 + \alpha_3 x_3 + \alpha_4 x_4 \quad (2)$$

$f$ : Ratio of expressway use
X1: Transport distance (km)
X2: Facility type from which freight is shipped,
where refrigerator warehouse = 1, other facility = 0.
X3: Delivery time specification, where hour specified =1, not specified = 0.
X4: Freight lot (ton)

Table 5 Estimation results

| coefficients | Estimate | Std. Error | Z value | Signif. |
|---|---|---|---|---|
| (Intercept) | -0.5181533 | 0.0522113 | -9.924 | *** |
| Transport distance (km) | 0.0050396 | 0.0001602 | 31.466 | *** |
| Facility type from which freight is dispatched | 0.7231309 | 0.0535702 | 13.499 | *** |
| Delivery time specification | 0.207155 | 0.0513031 | 4.038 | *** |
| Freight lot (ton) | -0.1531429 | 0.0199895 | -7.661 | *** |

AIC: 10858

Number of Fisher Scoring iterations: 5

Signif. codes: 0 '***' 0.001 '**' 0.01 '*' 0.05 '.' 0.1 ' ' 1

Null deviance: 11874 on 9352 degrees of freedom

Residual deviance: 10850 on 9349 degrees of freedom

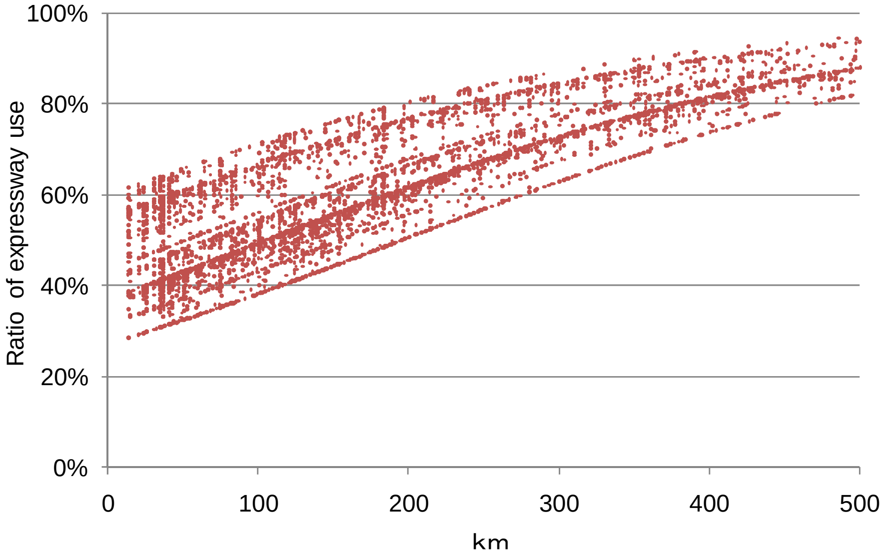



Figure 10 Plots of figures predicted by the model

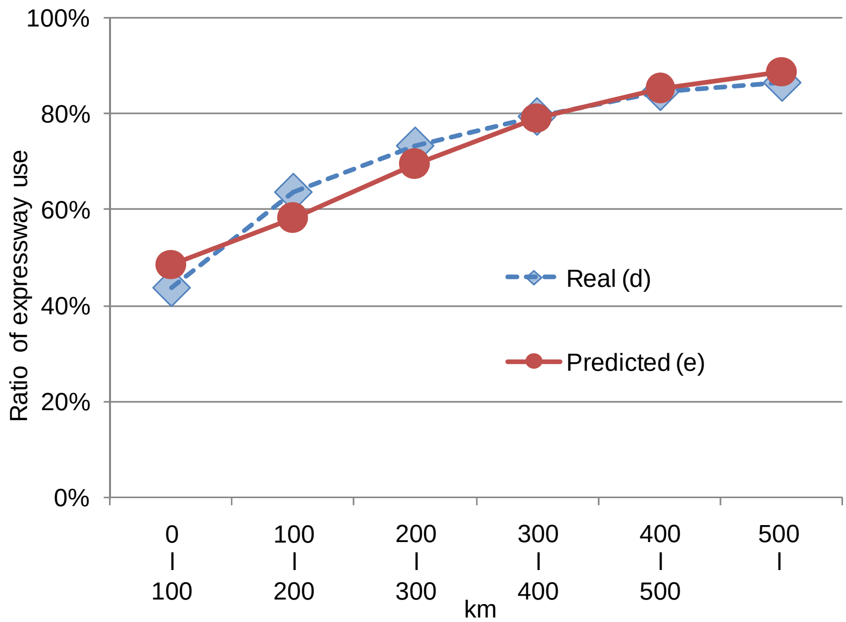



Figure 11 Average ratio of expressway use (real figures / predicted figures)

Table 6 Average ratio of expressway use (real figures / predicted figures)

| km | Total (a) | Number of samples Expressway Use (b) | | Ratio of expressway use (b/a) | | Difference | |
|---|---|---|---|---|---|---|---|
| | | Real | Predicted※ | Real (d) | Predicted (e) | e-d (g) | g/d |
| 0-100 | 2,509 | 1,095 | 1,219.9 | 43.6% | 48.6% | 5.0% | 0.114 |
| 100-200 | 2,259 | 1,437 | 1,310.1 | 63.6% | 58.0% | -5.6% | -0.088 |
| 200-300 | 1,230 | 901 | 851.7 | 73.3% | 69.2% | -4.0% | -0.055 |
| 300-400 | 881 | 698 | 697.2 | 79.2% | 79.1% | -0.1% | -0.001 |
| 400-500 | 433 | 366 | 368.4 | 84.5% | 85.1% | 0.6% | 0.007 |
| 500- | 2,041 | 1,762 | 1,811.8 | 86.3% | 88.8% | 2.4% | 0.028 |
| Total | 9,353 | 6,259 | 6,259.0 | 66.9% | 66.9% | - | - |

※Accumuration of the predicted f

## 5. PREDICTION OF CHANGES IN THE RATIO OF EXPRESSWAY USE

We carried out simulations using the model obtained in the last chapter to predict future changes in the ratio of expressway use in the following three cases;

Case 1: Increase in transport distance

Average transport distance and share of freight trucks even in long distance categories have increased, as shown in Figure 12 and Figure 13. This trend is expected to continue due to the advantages freight trucks have in door-to-door transport and at-any-time transport in consideration of demand for more sophisticated freight service. Case 1 simulates conditions in which the transport distance of each freight shipment is increased by 50%.

Case 2: Increase in the number of freight with delivery time specified

As shown in Chapter 3 (2), an increase in the percent of freight with delivery time specified is expected. Case 2 simulates conditions in which each freight shipment is hour specified.

Case 3: Decrease in freight lot

As shown in Chapter 3 (3), the trend of decreasing freight lot is expected to continue. Case 3 simulates conditions in which freight lot decreases to 50%.

The results of the simulations are shown in Figure 14 and Table 7, where Case 0 represents the figures predicted from the actual records in the previous chapter. The predicted ratio of expressway use for Cases 1, 2, and 3 were up, 7.4, 2.6, and 0.7 points from Case 0, respectively. Case 3 yielded only a slight change; change in lot does not affect the ratio of expressway use as much as transport distance and delivery time specification.

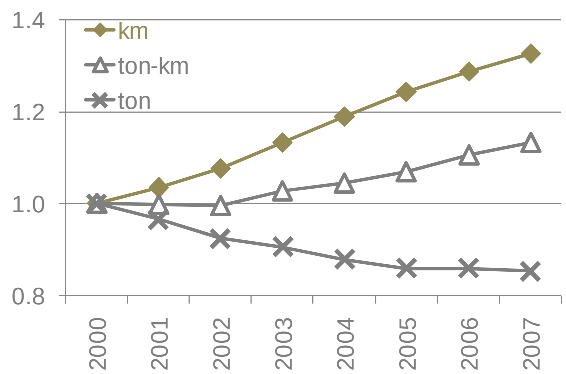



Figure 12 Average transport distance of freight trucks (ratio to the year 2000 figure)

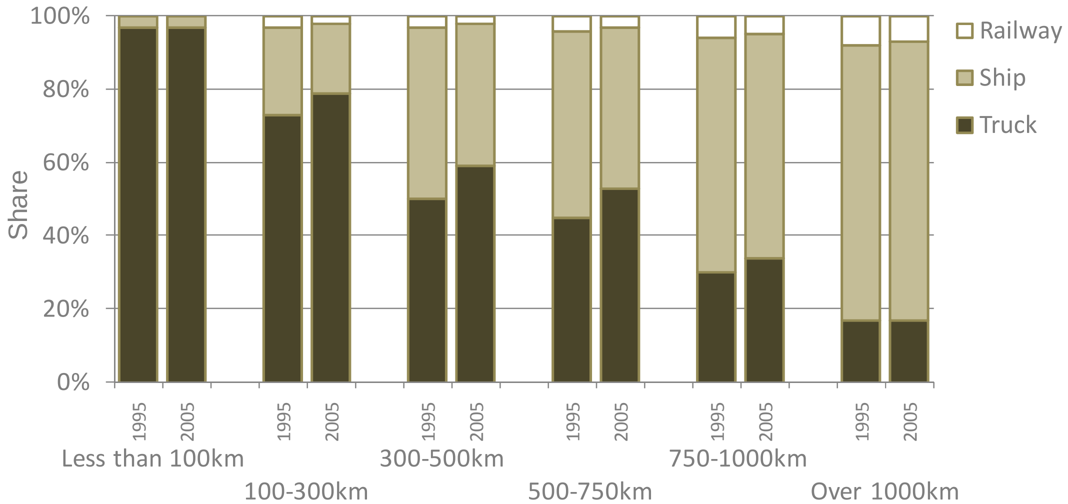



Figure 13 Share of means of transportation by distance category in the years 1995 and 2005

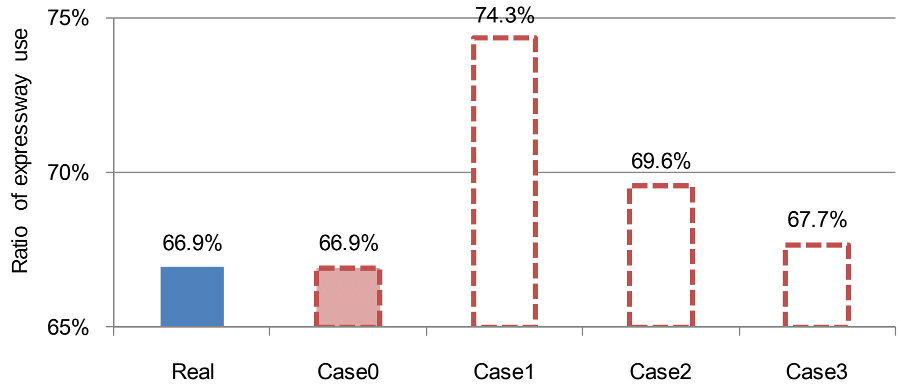



Figure 14 Ratio of expressway use (simulation results)

Table 7 Ratio of expressway use (simulation results)

| km | Number of sample | | Accumuration of the predicted f (b) | | | | Difference from Case0 | | |
|---|---|---|---|---|---|---|---|---|---|
| | Total (a) | Real | Case0 | Case1 | Case2 | Case3 | Case1 | Case2 | Case3 |
| 0-100 | 2,509 | 1,095 | 1,219.9 | 1,297.4 | 1,303.8 | 1,247.3 | 77.6 | 83.9 | 27.4 |
| 100-200 | 2,259 | 1,437 | 1,310.1 | 1,496.1 | 1,378.9 | 1,330.8 | 186.0 | 68.7 | 20.7 |
| 200-300 | 1,230 | 901 | 851.7 | 987.2 | 889.2 | 859.3 | 135.5 | 37.6 | 7.7 |
| 300-400 | 881 | 698 | 697.2 | 792.4 | 718.7 | 701.3 | 95.3 | 21.5 | 4.1 |
| 400-500 | 433 | 366 | 368.4 | 409.1 | 376.4 | 370.2 | 40.7 | 8.0 | 1.8 |
| 500- | 2,041 | 1,762 | 1,811.8 | 1,969.8 | 1,839.0 | 1,819.8 | 158.0 | 27.2 | 8.1 |
| Total | 9,353 | 6,259 | 6,259.0 | 6,952.0 | 6,505.9 | 6,328.8 | 693.0 | 246.9 | 69.8 |

| km | Ratio of expressway use (b/a) | | | | | Difference from Case0 | | |
|---|---|---|---|---|---|---|---|---|
| | Real | Case0 | Case1 | Case2 | Case3 | Case1 | Case2 | Case3 |
| 0-100 | 43.6% | 48.6% | 51.7% | 52.0% | 49.7% | 3.1 | 3.3 | 1.1 |
| 100-200 | 63.6% | 58.0% | 66.2% | 61.0% | 58.9% | 8.2 | 3.0 | 0.9 |
| 200-300 | 73.3% | 69.2% | 80.3% | 72.3% | 69.9% | 11.0 | 3.1 | 0.6 |
| 300-400 | 79.2% | 79.1% | 89.9% | 81.6% | 79.6% | 10.8 | 2.4 | 0.5 |
| 400-500 | 84.5% | 85.1% | 94.5% | 86.9% | 85.5% | 9.4 | 1.8 | 0.4 |
| 500- | 86.3% | 88.8% | 96.5% | 90.1% | 89.2% | 7.7 | 1.3 | 0.4 |
| Total | 66.9% | 66.9% | 74.3% | 69.6% | 67.7% | **7.4** | **2.6** | **0.7** |

## 6. CONCLUSION

By analyzing the data obtained from the Commodity Flow Census, we discussed the relationship between freight characteristics and the ratio of expressway use and discovered the following:

- The ratio of expressway use for freight shipped from refrigerator warehouses and for freight with delivery time specified is higher than that of other freight.
- The following four factors have a statistically significant influence on freight truck route selection in terms of the ratio of expressway use; the four factors are (1) transport distance, (2) facility type from which freight is shipped, (3) delivery time specification, and (4) freight lot.

The accuracy ratio of the model was 0.72; this was not very high, indicating the need to improve the model. In the future, we must consider other characteristics that serve as factors that can influence the ratio of freight truck expressway use and incorporate them into the model.

## REFERENCES

1) T. Kono. “Better use of space for measure for freight transport”. *IATSS review vol. 30*. No. 4. 2005
2) MLIT. Freight and passenger flow research. 2007

# STUDIES ON TRAVEL TIME RELIABILITY FOR INTERCITY EXPRESSWAY UNDER VARIOUS EVENTS

**Takeshi Mastsushita**

*Traffic Division Maintenance and Operation Department,*

*West Nippon Expressway Company Ltd.*

*Dojima Avanza 19F, 1-6-20 Dojima, Kita-ku, Osaka, 530-0003, Japan*

*t.matsushita.ab@w-nexco.co.jp*

## ABSTRACT

NEXCO-West, one of the intercity toll road operators in Japan, provides travel time information on its expressways. Currently, travel time information is provided in limited areas, and those information has less priority in case of traffic accident. Moreover, many signboards only provide information on congestion length, during the congestion caused by traffic accidents.

As a result, many customers complain about lack of travel time information in case of traffic jams and traffic accidents, since they spend more time than they have expected.

Using the traffic speed data measured by vehicle sensors near the major traffic congestion points, this paper describes results of analysis on distribution of travel time during various events occurred in the past year. The paper also reviews travel time an indicator called Buffer Time (*BT*), and suggests more comprehensive travel time indices to drivers compared to *BT*.

### KEY WORDS

BT, travel time reliability

## 1. INTRODUCTION:

West Nippon Expressway Company (hereafter "NEXCO-West") is operating and managing expressways over 3,300km in length in the western part of Japan.
Approximately 2.3 million traffics are on its expressways every day. (Apr-2009)
We are providing real-time traffic information by various means such as Variable Message Signboard (hereafter "VMS") at highway entrance and exit, toll gates, or JCTs, and radio broadcasting. Furthermore, we are providing the same information by use of cellular phone using internet function.
It is true that real-time information is informative and helpful for driver's who are about to leave their home or office, but these data cannot be useful for driver's decision making on their travel plan for his future trip (i.e., tomorrow or maybe next week).
We are providing information of congestion forecast, and estimated travel time to destination IC for 5 months by use of the record of past 3 years at areas in heavy inbound traffic congestion.
Therefore, these data are not useful once the incidents occurred.
Especially, people who need to catch international flight must be very punctual, and travel time reliability is very important. However, so far the travel time information has not been enough to meet their demand.
In this paper, results of analyses concerning travel time reliability is described, by calculating the required time for any departure time with or without incident.
Some case studies were performed using sample sections of 30km to 100km interval including the areas with frequent traffic congestion.

## 2. METHOD OF ANALYSIS

### 2.1 Target of analysis

For our case-study, 5 sections shown in Table 1 was used as sample cases. They were selected based on their frequent occurrence of traffic congestion between 2008-Jan-1 to 2008-Dec-31 (365days).
In this paper, results of case studies for section #3 and #4 is described.

Table 1 Target of analysis

| № | Road Name | Section | | | Length | Congestion Number of Times (2007) |
|---|---|---|---|---|---|---|
| 1 | Meishin-EXPWY | Suita | ～ | Nishinomiya | 21.3 km | 174 |
| 2 | Kinki-EXPWY | Suita | ～ | Matsubara | 27.5 km | 1551 |
| 3 | Chugoku-EXPWY | Chugoku-Suita | ～ | Nishinomiya-kita | 28.9 km | 769 |
| 4 | Hanwa-EXPWY | Matsubara | ～ | Gobo | 101.9 km | 660 |
| 5 | Nishimeihan-EXPWY | Matsubara | ～ | Tenri | 27.2 km | 465 |

### 2.2 Calculating travel time by time-slice method

Time-slice method is a dynamic travel time estimation method to calculate travel time by considering elapsed time from departure and dynamically summing up each travel time.
The velocity data were provided from vehicle detectors located every 2km interval.
In this paper, departure time was set every 5 minutes, requiring 12 samples within 1 hour.
Travel time provided on VMS is the estimation at the time of departure.

Fig. 1 Conception of Time-slice method

| Time | Travel time every vehicle detectors (min) | | | | |
|---|---|---|---|---|---|
| | section A | section B | section C | section D | section E |
| 7:00 | 3 | 5 | 4 | 6 | 5 |
| 7:05 | 3 | 6 | 5 | 7 | 7 |
| 7:10 | 4 | 6 | 6 | 8 | 9 |
| 7:15 | 4 | 7 | 6 | 8 | 10 |
| 7:20 | 5 | 7 | 6 | 9 | 10 |
| 7:25 | 5 | 7 | 7 | 10 | 9 |
| 7:30 | 5 | 8 | 7 | 10 | 8 |

Travel time provided on VMS (23min.)

Travel time by Time-slice method (31min.)

**2.3 The classification of travel time**

We analyzed the decrease of travel time reliability at the time of incidents (i.e., traffic accidents or road work) by classifying calculated travel time with or without incident using the past record of congestion data.

## 3. TRAVEL TIME RELIABILITY ANALYSIS BY THE DAY

We calculated a reliability index by the days of the week with a normal time reliability index without the incident to analyze the change of the travel time reliability by the difference of the day.

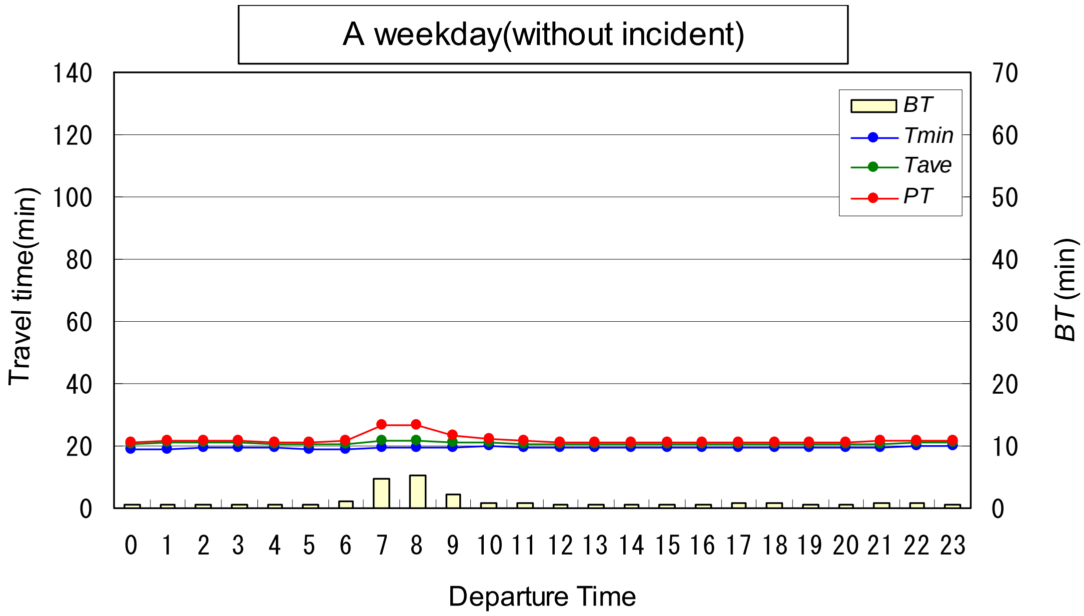
### 3.1 Section #3 Chugoku Expressway Chugoku-Suita – Nishinomiya-kita (30km)

This section is experiencing frequent congestion in almost whole section on weekend and seasonal traffic congestion period.

#### 3.1.1 A weekday

Travel time is increased from 7:00 am to 9:00 am, and the peak point is shown around 8:00 am.
Around the peak period, Average travel time (*Tave*) is 21 minutes (2 more minutes compared to the 5%-tile travel time (*Tmin*) of 19minutes).
95%-tile travel time (Planning Time hereafter *PT*) was about 27 minutes (8 minutes increase from *Tmin*) Buffer time (*BT*) was 5 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 3 minutes for each time. *BT* for each hour is almost limited within one minute, (See Fig-2)

Fig. 2 Travel time ( a weekday) without incident

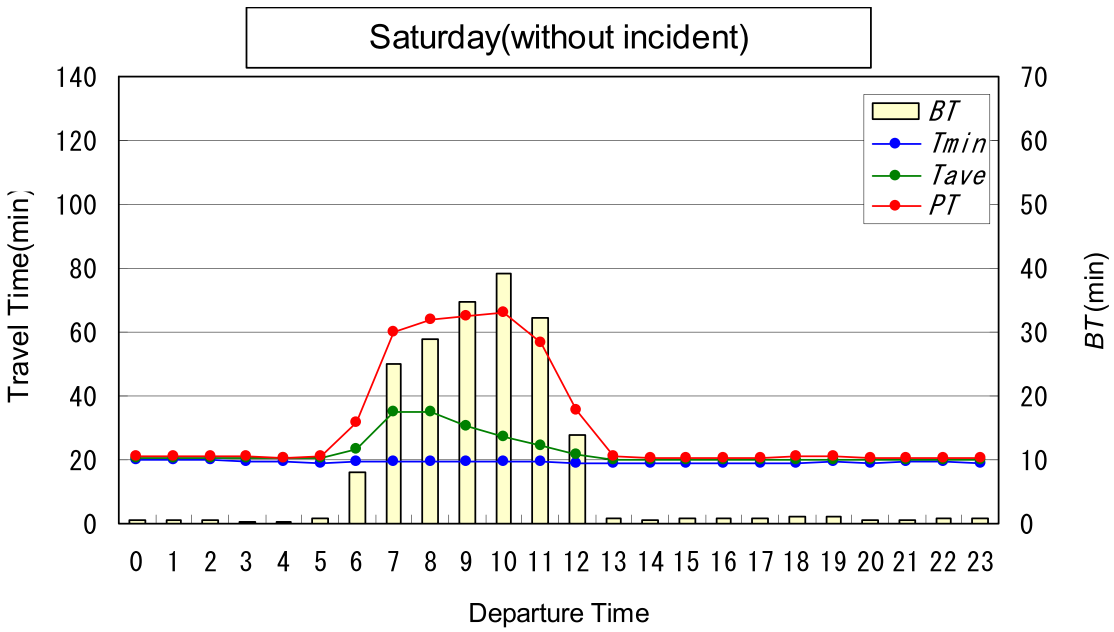



**3.1.2 Saturday**

Travel time is increased from 6:00 am to 12:00 am, and the peak point is shown around 10:00 am. Around the peak period, Average travel time (*Tave*) is 27 minutes (7 more minutes compared to the 5%-tile travel time (*Tmin*) of 20minutes). *PT* was about 66 minutes (46 minutes increase from *Tmin*). Buffer time (*BT*) was 39 minutes.

The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 2 minutes for each time. *BT* for each hour is almost limited within one minute, (See Fig-3)

Fig. 3 Travel time (Saturday) without incident

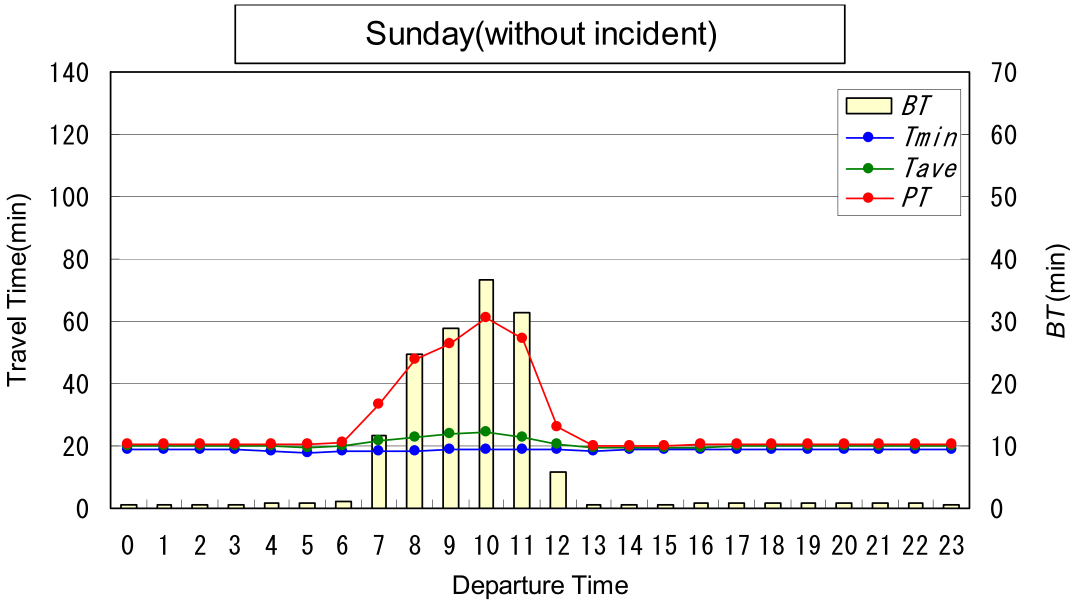



**3.1.3 Sunday / Holiday**

Travel time is increased from 7:00 am to 12:00 am, and the peak point is shown around 10:00 am. Around the peak period, Average travel time (*Tave*) is 24 minutes (5 more minutes compared to the 5%-tile travel time (*Tmin*) of 19minutes). *PT* was about 61 minutes (42 minutes increase from *Tmin*). Buffer time (*BT*) was 37 minutes.

The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 3 minutes for each time. *BT* for each hour is almost limited within one minute, (See Fig-4)

Fig. 4 Travel time (Sunday / Horiday) without incident



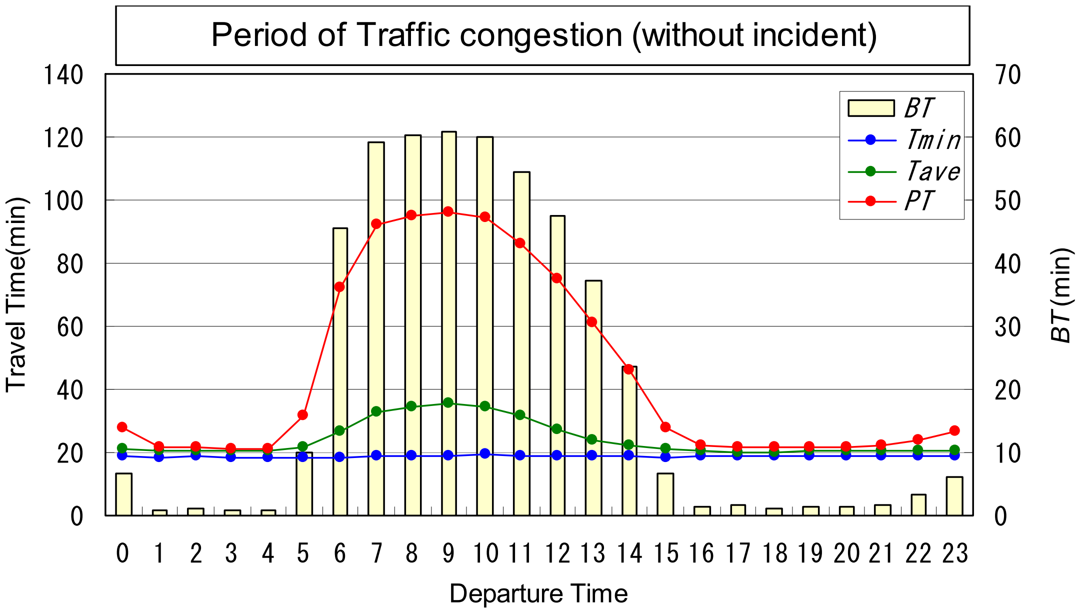

**3.1.4 Period of Traffic congestion**

Travel time is increased from 5:00 am to 3:00 pm, and the peak point is shown around 9:00 am.

Around the peak period, Average travel time (*Tave*) is 35 minutes (16 more minutes compared to the 5%-tile travel time (*Tmin*) of 19minutes). *PT* was about 96 minutes (77 minutes increase from *Tmin*). Buffer time (*BT*) was 61 minutes.

The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 3 minutes for each time. *BT* for each hour is almost limited within 2 minutes, (See Fig-5)

Fig. 5 Travel time (Period of Traffic congestion) without incident



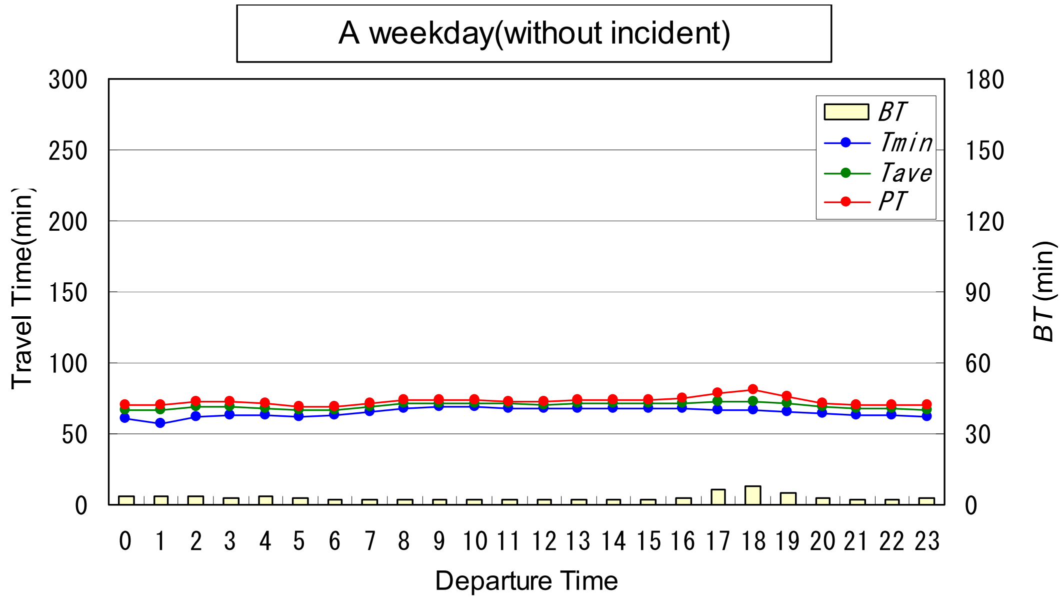
### 3.2 Section #4 Hanwa Expressway Gobo – Matsubara (100km)

This section is experiencing frequent congestion about 20 km on weekend and seasonal traffic congestion period.

#### 3.2.1 A weekday

In comparison with other days, a change is small for weekday travel time. (See Fig-6)

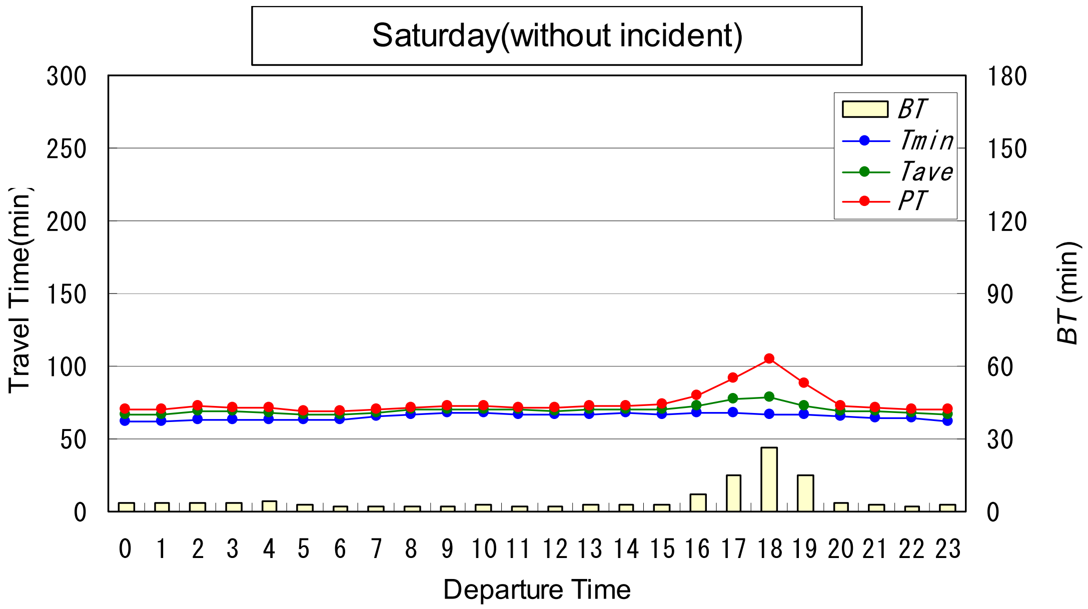
Fig. 6 Travel time (A weekday) without incident




#### 3.2.2 Saturday

Travel time is increased from 4:00 pm to 7:00 pm, and the peak point is shown around 6:00 pm.
Around the peak period, Average travel time (*Tave*) is 78 minutes (11 more minutes compared to the 5%-tile travel time (*Tmin*) of 67minutes). *PT* was about 105 minutes (38 minutes increase from *Tmin*). Buffer time (*BT*) was 27 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 7 minutes for each time. *BT* for each hour is almost limited within 3 minutes, (See Fig-7)

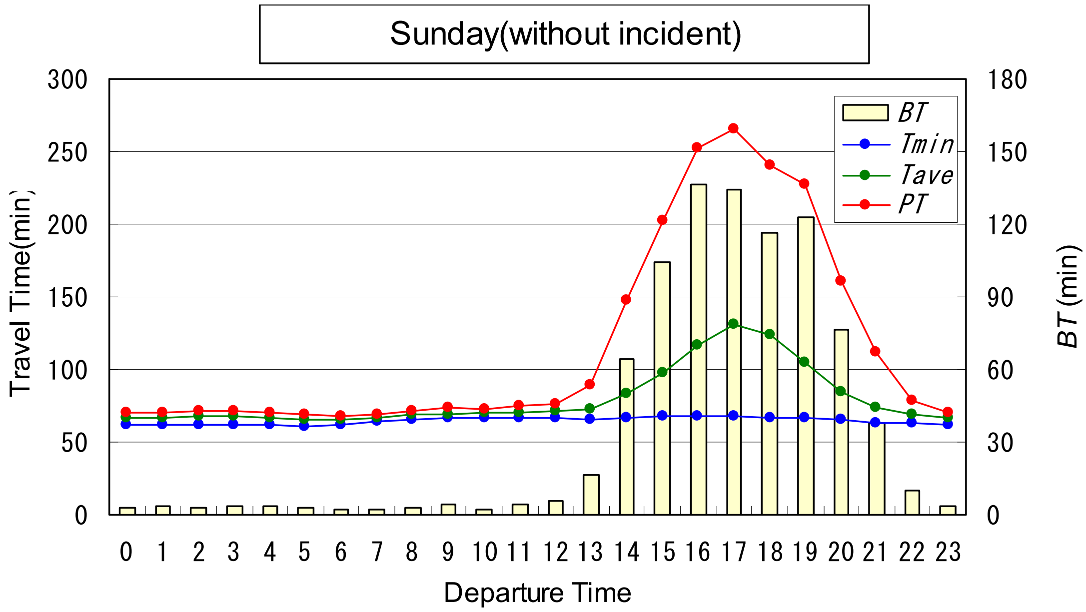
Fig. 7 Travel time (Saturday) without incident



### 3.2.3 Sunday / Holiday

Travel time is increased from 12:00 am to 10:00 pm, and the peak point is shown around 5:00 pm.
Around the peak period, Average travel time (*Tave*) is 131 minutes (64 more minutes compared to the 5%-tile travel time (*Tmin*) of 67minutes). *PT* was about 265 minutes (198 minutes increase from *Tmin*). Buffer time (*BT*) was 134 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 8 minutes for each time. *BT* for each hour is almost limited within 5 minutes, (See Fig-8)

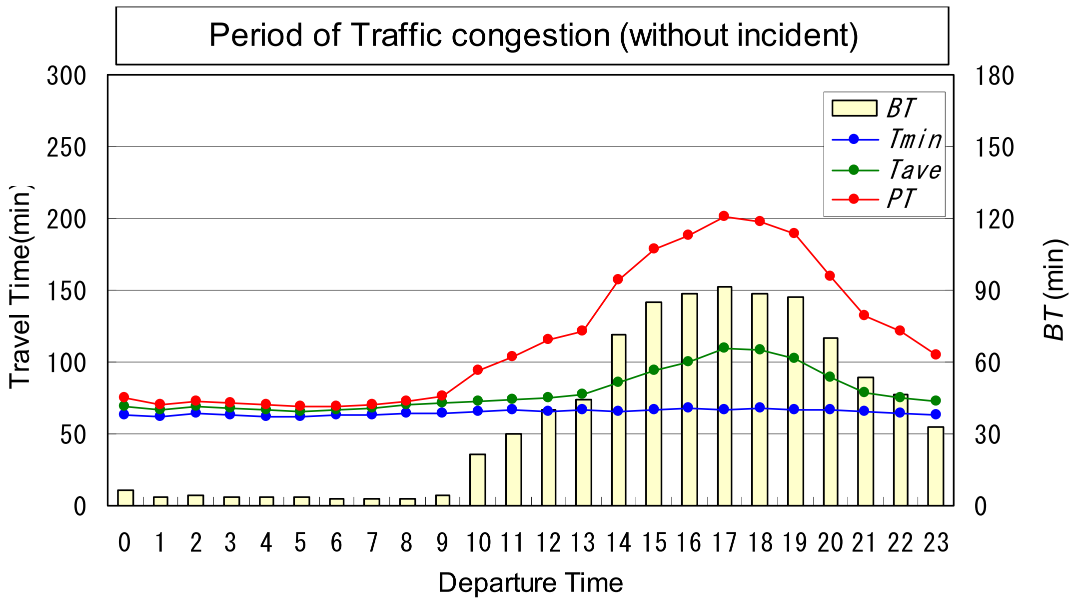
Fig. 8 Travel time (Sunday / Horiday) without incident




### 3.2.4 Period of Traffic congestion

Travel time is increased from 9:00 am to 1:00 am, and the peak point is shown around 5:00 pm.
Around the peak period, Average travel time (*Tave*) is 110 minutes (43 more minutes compared to the 5%-tile travel time (*Tmin*) of 67minutes). *PT* was about 201 minutes (134 minutes increase from *Tmin*). Buffer time (*BT*) was 92 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 9 minutes for each time. *BT* for each hour is almost limited within 4 minutes, (See Fig-9)

Fig. 9 Travel time (Period of Traffic congestion) without incident



## 4. TRAVEL TIME RELIABILITY ACCORDING TO THE INCIDENT PRESENCE

In order to figure out the deteriorating condition of travel time reliability with incidents, we compared Planning Time (*PT*) at normal time (without incident) and the case with incident. (See Table-2)
In addition, 2 sections above mentioned, we compared travel time reliability according to incident presence. (See Fig-10,11)

Table 2. Travel Time reliability according to the incident presence

| № | Road Name | Section | | | Length | A Day | Departure Time | *PT* (min) | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Normal (A) | Incident (B) | (B)-(A) |
| 1 | Meishin-EXPWY | Suita | ～ | Nishinomiya | 21.3 km | Period of Traffic congestion | 11:00am | 25.9 | 87.1 | 61.2 |
| | | | | | | Weekday | 9:00am | 15.7 | 64.2 | 48.5 |
| 2 | Kinki-EXPWY | Suita | ～ | Matsubara | 27.5 km | Saturday | 8:00am | 26.9 | 119.2 | 92.3 |
| | | | | | | Period of Traffic congestion | 8:00am | 31.3 | 145.8 | 114.5 |
| 3 | Chugoku-EXPWY | Chugoku-Suita | ～ | Nishinomiya - kita | 28.9 km | Weekday | 4:00pm | 22.7 | 118.3 | 95.6 |
| | | | | | | Period of Traffic congestion | 1:00pm | 60.9 | 110.0 | 49.1 |
| 4 | Hanwa-EXPWY | Matsubara | ～ | Gobo | 102 km | Sunday/Horiday | 5:00pm | 265.2 | 381.5 | 116.3 |
| | | | | | | Saturday | 8:00am | 98.9 | 326.6 | 227.7 |
| 5 | Nishimeihan-EXPWY | Matsubara | ～ | Tenri | 27.2 km | Weekday | 6:00pm | 20.7 | 132.7 | 112.1 |
| | | | | | | Weekday | 8:00am | 19.9 | 128.9 | 109.0 |

Fig. 10 Travel time (Weekday)

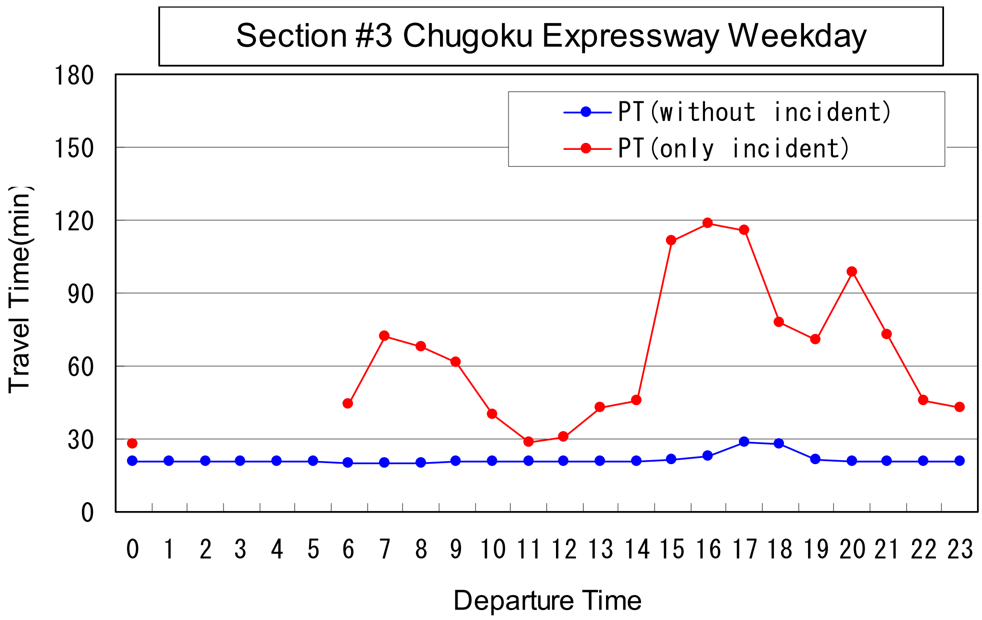
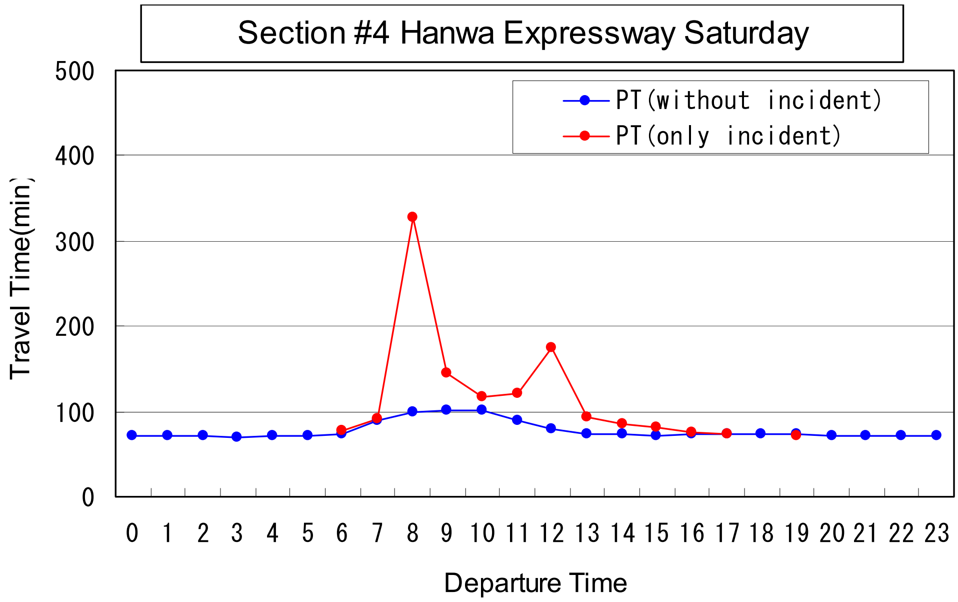


Fig. 11 Travel time (Saturday)




*PT* with incident is larger than normal time each sections and each day. If many drivers are caught in a jam caused by incident, they are usually forced to spend more travel time than expected.
Especially, maximum value of *PT* with incident concerning the section No.4 (Hanwa Expwy from Matsubara to Gobo) is 327 minutes, drivers are forced unexpected travel time for 228 minutes in comparison with *PT* at normal time (99 minutes).
Maximum value of *PT* with incident is marked on various days, and the characteristic of day is almost negligible, the phenomenon may be caused anytime.
However, probability of encountering incident will be at most 10% in the section we analyzed, Whether the risk is acceptable or not depends on the drivers' perspective.

## 5. CONCLUSION

Currently, travel time reliability is not familiar with highway uses.
In order to establish a travel time estimation system like the time schedule of the railway, we must investigate users' acceptable range of the reliability. For example, we have to ask the truck drivers' perspective of the Buffer Time (95%-tile travel time or 80%-tile travel time or others) in order to minimize their risks.
At the same time, we must consider how we could advertise to the public and let them get these data easily. For example, we can recommend drivers the departure time at prearranged accommodation by working together with lodging reservation website.
In this study, the analysis was performed with the past 1 year data. We have to improve reliability by analyzing more data.

# A 3-D MODEL FOR OPTIMAL ALIGNMENT SEARCH SYSTEM OF HIGHWAY DESIGN BY EXTENDED DIGITAL MAPPING DATA

**Seiichi KUSANO[1], Hidetoshi TAKAHASHI[2], Motoya YAMASAKI[3], Toshio TERAO[4], Norio TASHIRO[5], Thea Hack LORENTZEN[6], Minoru IMAI[7], Takayuki FUKASE[8]**

*[1]Civil Engineer, Central Nihon Expressway Company Ltd.,2-18-19,Nishiki, Naka-ku, Nagoya-shi, Japan, s.kusano.ab@c-nexco.co.jp [2]Civil Enginner, Central Nihon Expressway Company Ltd.,4-3-1, Toranomon, Minato-ku, Tokyo, Japan, n.takahashi.ah@c-nexco.co.jp [3]Associate Professor, Faculty of Regional Environment Science, Tokyo University of Agriculture, 1-1-1, Sakuragaoka, setagaya-ku, Tokyo, Japan, m3yamasa@nodai.ac.jp [4]Road Group, NEWJEC Inc., 2-3-20, Honjo-Higashi, Kita-ku, Osaka, Japan, teraots@newjec.co.jp [5]Technical Support Group, FORUM8 Co.,Ltd., 2-1-1, Kamimeguro, Meguro-ku, Tokyo, Japan, tashiro@forum8.co.jp [6]Road Support Group FORUM8 Co.,Ltd., 2-1-1, Kamimeguro, Meguro-ku, Tokyo, Japan, thea@forum8.co.jp [7]Transportation Planning Team, NEWJEC Inc., 1-12-13 Shin-Ohashi, Koto-ku, Tokyo, Japan, imaimn@newjec.co.jp [8]Road Group, NEWJEC Inc., 1-12-13 Shin-Ohashi, Koto-ku, Tokyo, Japan, fukasetk@newjec.co.jp*

**Abstract:**

In this study, we have made anThis study is an attempt to link OHPASS, Optimal Highway Path Automatic Search System, and three-dimensional (3D) CAD by including the attributes of information such as terrains, geological features, and environments within extended digital mapping (DM) data. In addition, we have constructed a system that links with 3D CAD, and enables output 3D data to be viewed and evaluated on VRas a virtual reality (VR) model. By In linking with 3D VR, its effective use is can be expected in the system can be used for road alignment and landscape planning and evaluation, as well as for presenting project proposals to a wider audience and road structure evaluation at the time of planning road alignment, consensus building with the parties concerned, and at explanatory meetings.

***Key Words :*** *3-D CAD, road design, virtual reality, extended digital mapping data*

## 1. INTRODUCTION

Many studies have been done on the construction of ideal highway alignments that satisfy various demands such as costs and restrictions, and they have contributed to the development of automated models for optimizing highway alignment. As a first step, in developing an automatic optimization model, it was necessary to determin main costs as well as, develop efficient algorithms, and link them to the actual Geographic Information System (GIS). Here "Main costs" refers to the expenditures that make up a high proportion of the total cost. Models for highway alignment optimization seeked to optimize highway alignments by minimizing the total cost.[1][2] It is also suggested that the following characteristics are essential for effective highway alignment optimization models [1] :

(1) Examines all the main costs and variable costs,
(2) Formulates all the important restrictions,
(3) Generates realistic alignments,
(4) Capable of treating alignments accompanied with backward bending,
(5) Optimizes horizontal and vertical alignments at the same time,

(6) Finds the most appropriate solution on or almost on a worldwide scale,
(7) Has efficient solution algorithms,
(8) Has a continuous search space,
(9) Considers the costs for intersections, interchanges, bridges, and tunnels,
(10) Automatically avoids inaccessible areas, and
(11) Has GIS compatibility.

For optimizing highway alignments, various conventional optimization methods have been used, including: the variational method, the dynamic planning method, numerical searching, the alignment planning method, and network optimization (Howard et al, 1968; Thomson and Sykes, 1998; Shaw and Howard, 1981, 1982; OECD, 1973; Turner and Miles, 1971; Turner, 1978; Athanassoulis and Calogero, 1973; Parker, 1977; Trietsch and Handler, 1985; Trietsch, 1987a, b; Hogan, 1973; Nicholson et al., 1976).[2][3][4] However, most of these methods lacked at least one of the highway alignment optimization model characteristics indicated above.

Genetic algorithms have proven to be effective for optimizing highway alignments, Particularly as the optimization of horizontal (planar) and vertical alignments can be performed effectively at the same time.[1][3] This is performed by searching for a better solution through continuous generation as well as by utilizing the whole search space without stopping at a local solution. This algorithm enables optimization of both horizontal and vertical alignments. It is possible to generate a flat and continuous alignment (e.g. not a corridor but an accurate linear shape), while directly linking the generated alignments to an actual GIS map.

The Optimal Highway Path Automatic Search System (OHPASS) has been created for the purpose of performing quantitative evaluation on a large number of alignments that are unable to be examined by conventional road design methods for highway alignment plans.[5][6][7] This study reports a new approach, which has never been published, that visualizes an optimized alignment by linking extended DM data using GIS to OHPASS, as well as by linking 3D-CAD to a VR system at the same time; i.e. constructing a system for highway alignment optimization and landscape simulation.

## 2. OVERVIEW OF EXTENDED DM (DIGITAL MAPPING)

### (1) Problems with using DM data

For road design, topographic map data is frequently used with road design CAD. Since DM (Digital Mapping) data doesn't require elevation data from benchmarcks, contour lines, or spot elevations, this elevation data will not be reflected in the DM data result, even when a topographic survey observes elevations. It has been problematic that DM data can't be utilized for road design. In order to solve this problem, specifications of extended DM data have been examined.

### (2) DM and extended DM

In "MLIT Work Regulations for Public Surveys" created by Ministry of Land, Infrastructure and Transport (MLIT) [8], specifications for DM data are provided under "Classification Standards for Acquisition of Digital Mapping Data" and "Specifications for Digital Mapping Data Files", which unify interpretations of data file specifications and clarify them. They also extend the specifications of DM data files for applications other than the products of digital terrain surveys, and put together a "Draft Code for Implementation of Extended Digital Mapping". Also, the "Draft Guidelines for Electronic Delivery of Survey Products" provid guidelines for creating a DM data file with these extended DM specifications, applying them as a form of electronic delivery for some of the products of fiducial point surveys and applied surveys in addition to the products of digital terrain surveys. In the "Draft Guidelines", the specifications provided under the "MLIT Work Regulations for Public Surveys" and the "Draft Code for Implementation of Extended Digital Mapping" are distinguished by naming the former "DM" and the latter "extended DM" respectively (Figure 1).

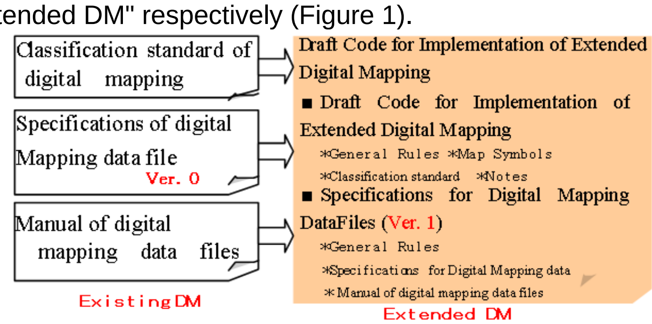


**Figure 1: DM and extended DM**

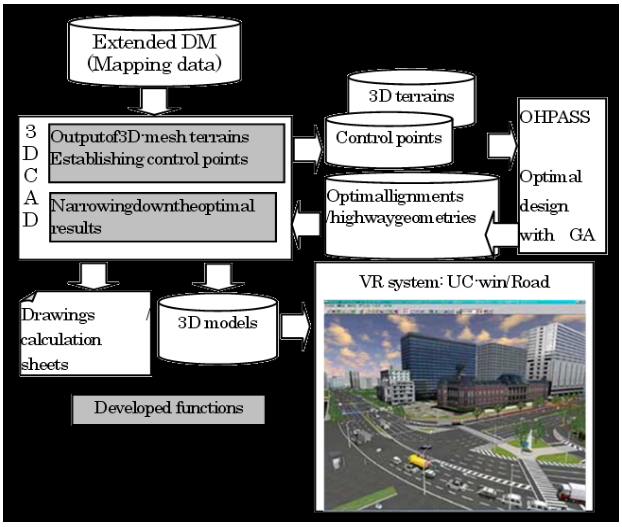



**Figure 2: System configuration**

The major difference between "DM" and "extended DM" is that the latter can be used for road design purposes. The primary changes includes: 1) unification of interpretation of use and minor modifications, and 2) application to applied surveys, etc.  Especially for extended DM, it has been specified that elevation values for contours, elevation points, etc. should be clearly described. Here, regarding terrain expression, there is a different type used by digital terrain models (DTM) in addition to using contours.  Work Regulations for Public Survey permit the use of both display types at the same time.

## 3. 3D-CAD, EXTENDED DM, AND A VR SYSTEM

### (1) System Configuration

We have developed a mechanism for creating 3D highway shapes and producing drawings with a data input and  creation function in OHPASS.  Figure 2 shows the system configuration diagram. This system has been developed based on AutoCAD Civil 3D 2007 (reffered to as Civil 3D), a civil engineering CAD software. The functions developed include a method for creating 3D terrain and control points that are input using Civil 3D, and a function to incorporate the calculated results back into Civil 3D.

Civil 3D has functions that include loading DM, creating 3D representations of terrain, road design (horizontal design, vertical design, and cross-sectional design).  Moreover, we attempt to link the system with UC-win/Road Ver.3.2 (produced by FORUM8 Co., Ltd.), one of the 3D-model VR systems. UC-win/Road is capable of providing various real-time presentations with CG after generating three-dimensional VR spaces, and has been utilized for examining landscapes, consulting for design, and presenting project briefings. It also provided the data exchange function with Civil 3D used in this study implemented, enabling linkage experiments to be performed without developing special software.

### (2) Data Conversion and the Establishment of Control Points: (OHPASS Input Interface)

In order to efficiently create input data for OHPASS, we developed an interface for converting terrain data, setting up control points, and then passing them into OHPASS.

a) Terrain data conversion function

In OHPASS, terrain is mesh datum. A 2m mesh is required for calculations, while a 20m mesh is required for display. For this reason, we have developed a function to create both 2m and 20m mesh files from a 3D surface model created with Civil 3D.  While Civil 3D normally provides an API (Application Program Interface) to export the elevation data of points that are specified in the 3D surface, we have increased the speed and efficiency of this process (e.g. by skipping unnecessary elevations when the file size for terrain data is too large).

b) Control point set-up function

In OHPASS, control points are expressed using lines (for railroads, rivers etc.) or areas (for

houses, prohibited planar areas, etc.). We have developed a function to add the information in OHPASS control points to the lines (unclosed polylines) or poligones (closed polyline) created by the standard CAD functions of Civil 3D.

**(3) Loading the Optimal Results (OHPASS Output Interface)**

We have developed a function that load the examined results of alignments obtained in OHPASS, and displays them as horizontal and vertical drawings. As a feature of OHPASS, it is possible to examine two or more alignments by changing design conditions; this time, a function for selecting and loading an alignment has been added so that loading can be performed with multiple results examined.

## 4. DEMONSTRATION EXPERIMENT (USING EXTENDED DM)

### (1) Purpose and Procedure

The advantages of using DM include time efficiency in the process of creating terrain data and establishing control points. Working hours are evaluated and, compared with a case where two dimensional terrain CAD data is used .

a) Labor saving in the process of creating 3D terrain data

As the contours include digital elevation data, time taken to create new three dimensional data can be saved. If height information other than contours is required, it is necessary to add the heights separately. However, for topographic maps with a scale of 1 to 2,500, which this system is designed for, it is assumed that the contours will be enough to achieve adequate design accuracy in three dimensional representation.

b) Labor saving in the process of establishing control points

In this study, we have set up control points assuming that extended DM data are classified into layers by map element type and then loaded into the CAD system.

Figure 3 shows the experiment procedure.

1. Road design experts organize evaluation items for the control points that should be considered in designing roads and for the design results.
2. Optimal design calculation is conducted for the actual object.
3. Results are summarized and evaluated.

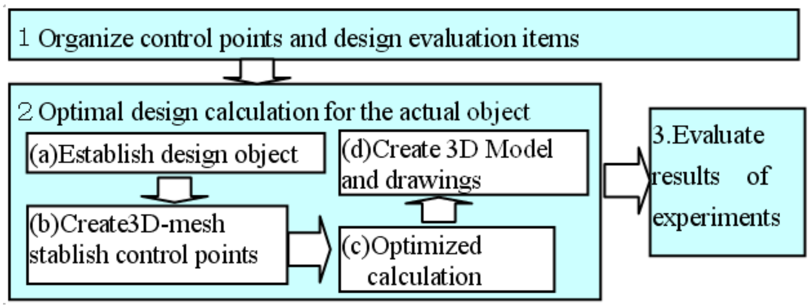



**Figure 3: Procedure for optimal highway design using extended DM**

In Step 2.("(a)" in Figure 3), difficulties were expected with selecting an object (for which a topographic map had been created with extended DM) out of past design objects. We selected an object for which a topographic map was organized as CAD data, assuming that the DM loading function implemented in this system could be simulated. In (b), we loaded the topographic map for the object selected in (a) into AutoCAD Civil 3D, and edited the layers etc. so that it would be loaded with the DM loading function. This was performed in order to simulate the DM loading function. Next, we created 3D terrains and control points using the newly developed functions. In (c), we specified geometric and earthwork conditions etc., and performed calculations for optimal highway design. In (d), we loaded the results of the optimal design calculation and created a 3D highway model and drawing.

### (2) Subject

The object selected as an experimental subject was a highway outline design of a partial section of an Arterial High-Standard Highway. Its overview is as follows:

(a) Designed daily traffic volume (H32): 16,300-21,800 vehicles/day
(b) Highway standard: 1st class, 2nd grade
(c) Design velocity: V=100km/h
(d) Design length: L=21.8km
(e) Structure width: W=20.5m, Four lanes on completion
(f) Design classification: highway outline design (B)
(g) Design year: 2003
(h) Planned as a completed section
(i) Design work has been performed with a plan of S=1/2500

In this demonstration, using the entire section (total length L=21.8km) of the above design object in the experiment would cause a large load to be applied to the computer. Therefore, in consideration of data handling and calculation time, we extracted a section of about 4km and used this as the subject of the experiment.

**(3) Creating 3D-Mesh Terrains and Establishing Control Points**

Even though it is preferable to conduct an experiment using data with extended DM specifications, as discussed in the experimental procedure, the terrain data for the experimental subject are not extended DM data but CAD data. The reason for this is that current specifications for extended DM are under development and there are no complete data sets in extended DM that can meet product specifications. In order to achieve the purpose of the experiment, we have loaded the terrain data and edited them to the envisioned format of the extended DM. Then, using the edited terrain data, we created a three dimensional terrain mesh and established control points. The procedure and the data created are explained below.

(a) Loading CAD data

Since the CAD data for the subject was in DWG format, the standard format for 3D CAD (Civil 3D), they were loaded without any problem.

(b) Editing the CAD data to simulate DM loading

We edited the loaded data in process (a) so that the data would be in the same condition as they would be in the case of loading extended DM. When using the DM loading function in Civil 3D, data are loaded after being classified into layers according to their map element type. This method is considered common for loading DM data into a CAD system.

In the original CAD data, only the contours had height information. We decided not to provide additional height information because it was assumed that the height data in the DM data output from existing survey work would be contours only. In the highway design under consideration, we had determined that the accuracy would be adequate when the height was given by contours.

(c) Establishing control points

Control points were established using the control point function explained in the above section titled "Linking 3D-CAD, extended DM, and a VR system."

**(4) Results of the optimal alignment search**

The optimal road alignment calculation system searches for the best alignment within a width of 200m from the left and right extremes of the initial alignment. This time, we searched for three routes, Plan A, B, and C.

Plan A - Northern route: This route passes through a northern area and is optimized with the actual design as the original alignment. It is used to compare the actual design (initial alignment) to the optimal calculation results.

Plan B - Central route: This route makes a southern detour around a natural woodland (environmental protection area) that coexists with a nearby population.

Plan C - Southern route: This route passes through the mountainous area in the south, avoids northern areas that are prone to landslides or considered environmental protection areas.

Figure 4 shows the schematic searching ranges for each plan. The upper drawing of Figure 4 shows the schematic searching ranges while the lower one shows the optimal routes searched through genetc algorithm.

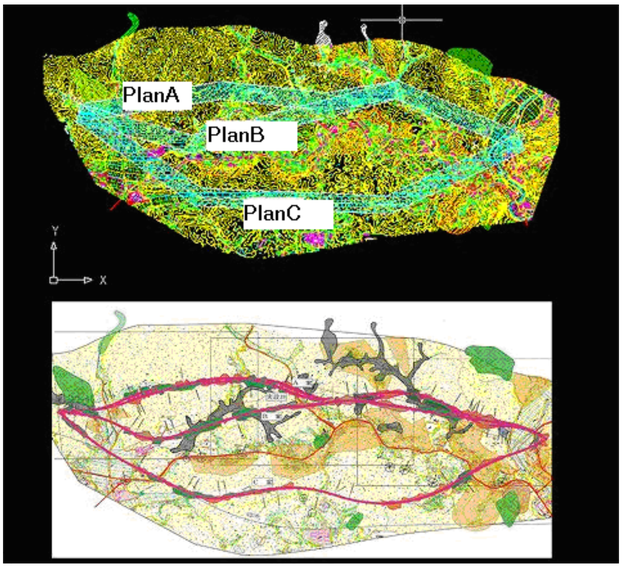



**Figure 4: Search range of each route**

Table 1 shows a cost comparison. Each of these three plans satisfy the geometric conditions. Plan A is the result of optimization by searching within a range 200m wide on both sides along the initial alignment and combining this with a cost cost analysis. As expected, when comparing the construction cost of the original plan to that of Plan A, Plan A costs less. However, there are still some problems. One problem is that the alignment of the actual design was partially a cross section of a dual carriageway, which was approximated by a single lane. Also, since a section of 4km was extracted out of the total length of 21.8km for the actual design, the volume of earth was not perfectly calculated.

**Table 1: Comparison of construction costs**

| Item | | Actualdesign | RouteA | RouteB | RouteC |
|---|---|---|---|---|---|
| Length (m) | Earthwork | 4,150 | 4,190 | 4,270 | 3,500 |
| | Bridge | 230+260+240+300 | 560+400 | 400+460 | 640+380 |
| | Tunnel | 0 | 0 | 0 | 900 |
| | Total | 5,180 | 5,150 | 5,130 | 5,420 |
| Project costs (1,000,000 yen) | Road (earth work) | 24,954 | 20,529 | 23,277 | 16,219 |
| | Bridge | 53,923 | 49,224 | 51,970 | 46,343 |
| | Tunnel | 0 | 0 | 0 | 39,396 |
| | Others | 17,263 | 17,144 | 17,632 | 14,274 |
| | Total | 96,140 | 86,897 | 92,879 | 116,232 |
| Alignment element | Minimum curve radius (m) | | 550 | 500 | 550 |
| | Minimum curve length (m) | | 230 | 259 | 308 |
| | Minimum transition curve (m) | | 550 | 500 | 550 |
| | Minimum transition curve length (m) | | 80 | 73 | 71 |
| | Steepest vertical gradient (%) | | 3.8 | 4 | |
| | Steepest vertical curve radius (m) | | 2486 | 12092 | 7446 |
| | Steepest vertical curve length (m) | | 154 | 472 | 268 |

In terms of costs, Plan A, is the least expensive and will be the best option among these three. Also, becase Plan A is the optimal result of the of the initial plan alignment, it verifies the validity of the actual design.

### (5) DM Resuts and Problems

This system provides very simple functions such as selecting CAD elements and specifying types of control points. A CAD element is an object such as a line or symbol that describes a feature on a drawing. Even this simple function may automatically reduce an operator's burden as it is easier to edit data that has been separated into layers (in order to establish control points) than to edit uncategorized CAD data. Although we did not measure time the following processes became possible:

(1) Showing only road layers, selecting them as a group, and setting up control points
(2) Hiding elements that have no relation to control points in order to improve visibility

In addition, with the development of more functions in the future, it will be possible to automatically convert a layer into a control point. In other words, when the map elements are drawn in designated layers, elements in each layer will be converted into control points together as a group.

Some problems that inhibit the effective use of the system have also been revealed.

a) Discrepancyof figures

There are some figures for which the figure type for extended DM is different from what is required as a control point. For example, a figure type for extended DM may contain data specifications for creating a map, while a control point for road design indicates things like cultural heritage areas or landslide prone areas. This corresponds mainly to the features represented by map symbols on the side of extended DM.

b)Lack of definition in extended DM

Naturally, some of the control points used in road design were not defined in the extended DM. These problems will be addressed by revising the specifications for the DM on a long-term basis. On a short-term basis, this can be addressed and by examining control points isually as well as by creating data by manual operations in CAD.

**5. DEMONSTRATION 2: LANDSCAPE EVALUATION THROUGH LINKAGE WITH A VR SYSTEM**

Although roadside landscape is one of the most important considerations for road design, OHPASS does not have a function to evaluate it. In this experiment, a person will judge the landscape using computer graphics (CG). We have linked OHPASS with a VR system and examined its uses and effects. The VR system configuration is described in “Linking 3D-CAD, Extended DM and a VR System: System configuration” section.

**(1) Automatic Generation of VR Spaces**

Terrain data are loaded from 3D data created within Civil 3D via a LandXML format file. Next, horizontal alignment data for highways (linear coordinates and parameters), vertical alignments (intersection pointst, height, VCL, etc.), and highway cross sections (width of road surface, slope gradient / bench spacing, etc.) are loaded. A three-dimensional space is created by combining all these data. 3D geometries are not loaded directly from 3D CAD data. Instead, VR software is used to load the cross-section data and build its design geometries. The geometry created by the VR software does not differ from from geometry created with 3D CAD software. When creating a 3D VR space, we have also used textures to represent details such as lane markings, road surface textures, and cutting / banking, which is not expressed in the process of 3D modeling. Figure 5 shows the link between the VR and Civil 3D systems.

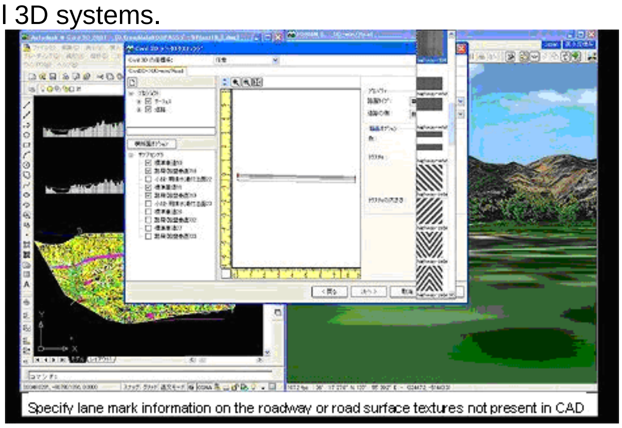



**Figure 5: Linkage of CAD and VR system**

**(2) Evaluation and Discussion**

We made a presentation showing an animation with real-time CG. Figure 6 shows the presentation screen. After presentation and examination the following results were achieved:

a) Landscape evaluation from a driver's perspective

The VR simulation can be utilized for examination and evaluation of safety issues such as driver viewing distance and visibility with different alignments. The geometry of cutting and banking and road structure has only a limited effect on drivers. For example, the sense of apprehension that a driver might feel when passing through a steeply cut terrain area can be evaluated to a certain extent. However, for examining the geometry of tunnel entrances, separate model creation apart from the described workflow is necessary for evaluation in the VR space.

b) Effects produced by substantial reduction in time for image generation

Streamlining the image creation process (by loading the extended DM in OHPASS to transferring it into 3D CAD and the VR system) can substantially reduce the working hours that would be required for image generation using a conventional method.

**Figure 6: Screen Example for presentation**

Table 2 shows the time required for these demonstration experiments. In this case demonstrations 1 and 2 are considered consecutive. Demonstration 1 was performed to search for an optimal alignment with OHPASS using the data converted into the specifications of extended DM. Demonstration 2 used the results produced by OHPASS and linked these with a VR system. In comparing the time required to generate alternative plans: (a) is equal. Yet this system makes it possible to generate alternative plans and preview them with the VR simulation in about 60 minutes, the total working time of steps (b), (c), and (d). For instance, when deciding between an embankment or an elevated structure, after calculating the alignments, the following procedure makes it possible to compare both plans in a short amount of time taking into account construction costs or volume of earthwork.

**Table 2: A comparison of working time**

| Work contents | | Working time required for the conventional method | Working time required for this system |
|---|---|---|---|
| (a) | Creating 3D terrains from DM and establishing control points | 240min | 240min |
| (b) | Optimal design calculation with OHPASS | 420min | 10min |
| (c) | 3D representation of the calculation results from OHPASS | 180min | 30min |
| (d) | Creating data with a VR system | 240min | 20min |
| Total | | 1,080min | 300min |

(1) Embankment geometry generation with a series of operations up to the VR system
(2) Establishment of an area to be made into an elevated structure as a control point for a "compulsory elevated section" using the function of OHPASS
(3) (b) Optimal design calculation using OHPASS --> (c) three-dimensional representation of the calculation results from OHPASS
(4) (d) Data creation using VR system and comparison with the result of (1)

## 6. CONCLUSIONS

In this study, we have attempted to link OHPASS, an optimal road alignment search program, with three-dimensional CAD. By creating 3D terrain from extended DM and using a function to establish control points, a basis for utilizing extended DM has been created. In addition, by enabling the optimal design calculation results to be loaded into 3D CAD, graphical representations of the design results have been linked with a VR system.

Using the developed system, we performed demonstrations with the extended DM. It was

concluded that using extended DM is effective for creating three-dimensional terrain models of the present conditions. In order to save time, these models are essential as input data for the optimal design system. Next, we conducted a linkage experiment with a VR system. It was confirmed that it is effective to evaluate landscapes in a short amount of time by utilizing extended DM as input data and linking the optimal highway alignment design system, 3D CAD, and VR system. We expect that a link between OHPASS and a VR system can be used effectively for landscape evaluation during highway alignment design stages, as well as for consensus building and presenting proposals. Also, as a link between UC-win/Road and driving simulator hardware has been created, we can expect that this research may also be useful for advanced driver assistant systems. Issues to be dealt with in the future include finding a way to provide height information for features other than contours in extended DM, creating shape required as control points, and dealing with items not considered in OHPASS.

**REFERENCES**


1. J.-C. JONG. Optimizing highway alignment with genetic algorithms. Ph.D. dissertation, University of Maryland, College Park, MD. 1998.

2. Manoj Kumar JHA. A geographic information systems-based model for highway design optimization. Ph.D. dissertation, University of Maryland, College Park, MD. 2000.

3. J.-C. JONG and Paul SCHONFELD. An Evolutionary model for simultaneously optimizing three-dimensional highway alignments. Transportation Research, Part B. 2003.

4. AASHOTO. A Manual on User Benefit Analysis of and Bus Transit Improvements. American Association of state Highway and Transportation Officials, Washington, Dc. 1977.

5. Masaya YAMASAKI, Teietsu HONGO, Hiroshi IMAMURA, Kazuo HIYANE, Jun LIO, Tomoyuki YATABE, and Seiichi KAGAYA. Basis study in optimal vertical design with digital terrain data and genetic algorithms. Infrastructure Planning, Collected papers, Vol.18， pp.139-148， 2001.

6. Masaya YAMASAKI, Teietsu HONGO, Kazuo. HIYANE, and Tomoyuki YATABE. Study of alignment models for optimizing highway alignments. Collected papers of Japan Society of Civil Engineers, No.758／IV-63， pp.57-69， 2004.4.

7. Masaya YAMASAKI, Teietsu HONGO, Kazuo HIYANE, Jun LIO and Tomoyuki YATABE. Genetic Algorithms Based Optimization of Three-Dimensional Alignments for Highways, Proceedings of the Third International Conference on Engineering Computational Technology, 2002.

8. Ministry of Land, Infrastructure, Transport and Tourism: "MLIT Work regulations for public survey". Japan Association of Surveyors, 2002.

# Effect of the Opening of Central Circular Shinjuku Route (Yamate tunnel): An Analysis

Atsushi Takehana, Masayuki Miura, Hiroshi Warita, Seiya Tazawa
*Metropolitan Expressway Company Limited*
*6-6-2 Nishi-Shinjuku, Shinjuku-ku, Tokyo, Japan*
*a.takehana18@shutoko.jp*

**ABSTRACT**

The Metropolitan Expressway's Central Circular Route runs innermost of the three major Ring Roads in Greater Tokyo. Central Circular Shinjuku Route (Yamate Tunnel), the 6.7-kilometer section of the route, was opened to traffic on December 22, 2007 in addition to its eastern and northern sections (33 kilometers).

The major role of this route is to allow drivers to avoid the central area district. It is expected that the inauguration of the route will contribute to ease the heavy traffic congestion in Central Tokyo.

In this paper, analysis on the effect of the Yamate Tunnel inauguration focusing on the service level of Metropolitan Expressway, based on various kinds of traffic data, is described. The results show that the tunnel inauguration produced significant improvements in the traffic conditions on Metropolitan Expressway, such as mitigation of the total traffic congestion, reduction of the travel time on certain routes, and decrease in traffic accidents.

## 1. INTORODUCTION

In the Tokyo area, construction has been completed on expressways linking Tokyo to other urban areas, including the Tomei Expressway, Chuo Expressway, and Tohoku Expressway, as well as portions of the Metropolitan Expressway system which link them radially. However, it is the Inner Circular Route (C1) only that circularly connects these expressways. As a result, traffic tends to become concentrated on the Inner Circular Route, which leads to chronic congestion.

As a fundamental solution to this problem of congestion, it is urgently necessary to complete additional beltways for the suitable diversion and dispersion of through traffic. Work is underway to construct an expressway network with three beltways: the Central Circular Route (C2) of the Metropolitan Expressway, which lies about 8 kilometers from central Tokyo; the Tokyo Outer Ring Road (Tokyo Gaikan Expressway), about 15 kilometers from central Tokyo; and the Metropolitan Inter-City Expressway (Ken-o-do), 40 to 60 kilometers from central Tokyo.

The Central Circular Route, closest of these to the city center, will be a 47-kilometer beltway.

In addition to its completed eastern and northern sections (about 26 kilometers), its western section, the Central Circular Shinjuku Route(Yamate Tunnel), will measure about 11 kilometers. And the Central Circular Shinagawa Route which is the last southern section is about 10 kilometers.

The first portion of the Yamate Tunnel, a 6.7 kilometer section linking the Route 4(Shinjuku Route) and Route 5(Ikebukuro Route), was opened in December 2007(Figure 1). The Yamate Tunnel runs about 30 meters underground along the path of the Loop Route 6, which is known as Yamate Street.

In this paper, we report on the effects that have been observed during the first year after the Yamate Tunnel was opened, based on data from vehicle detectors installed on the Metropolitan Expressway, accident data from traffic control, and Internet surveys.

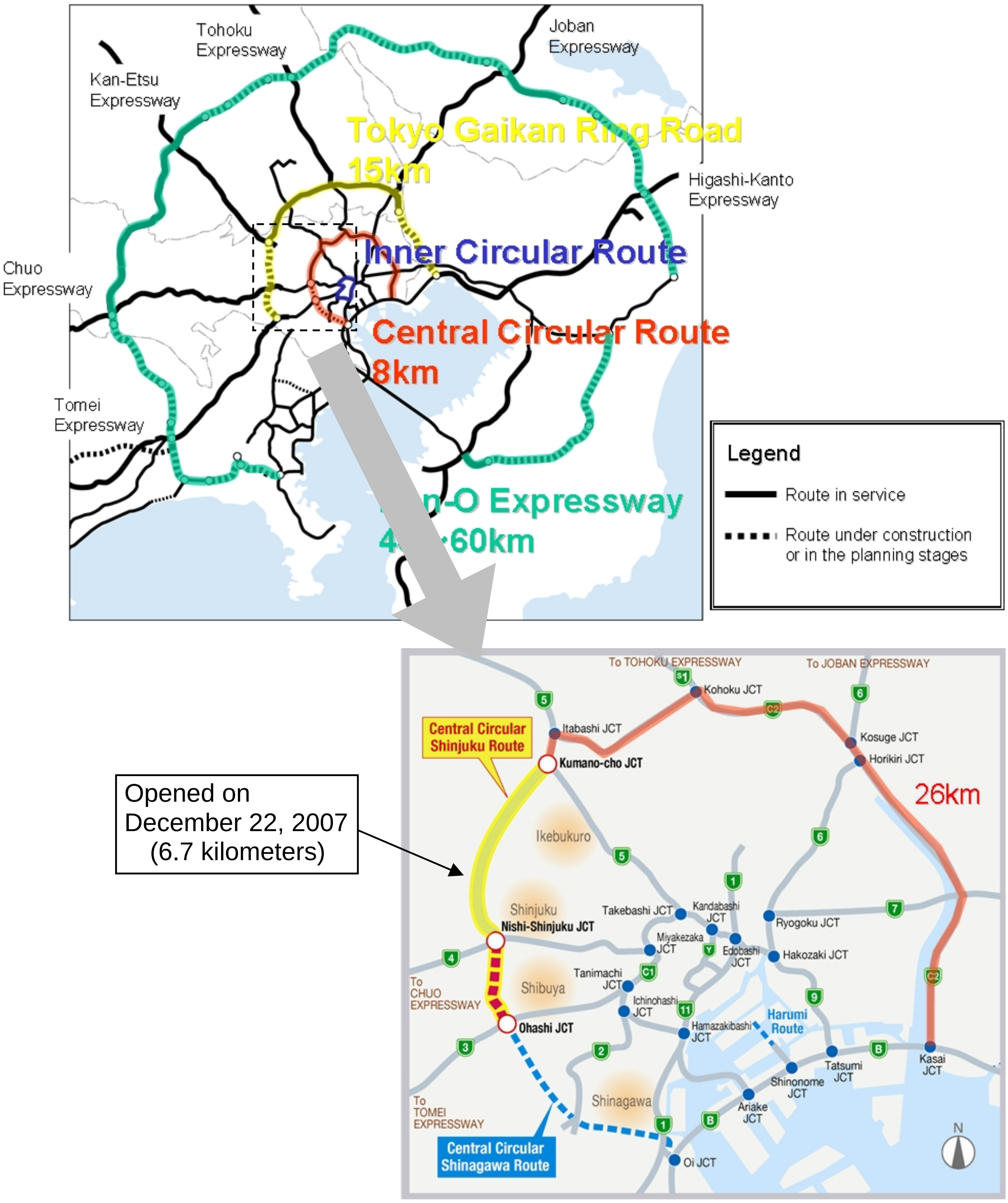



Figure 1. Expressway network in the Tokyo area

## 2. ROLE OF THE CENTRAL CIRCULAR ROUTE (C2)

As mentioned above, because of the delayed completion of beltways within the expressway network of the Tokyo area, traffic tends to become concentrated in one area. To travel from one radial route to another, it is generally necessary to take the Inner Circular Route (C1). About 60% of all traffic on C1 consists of through traffic, or vehicles which do not have destinations along C1 (Figure 2).

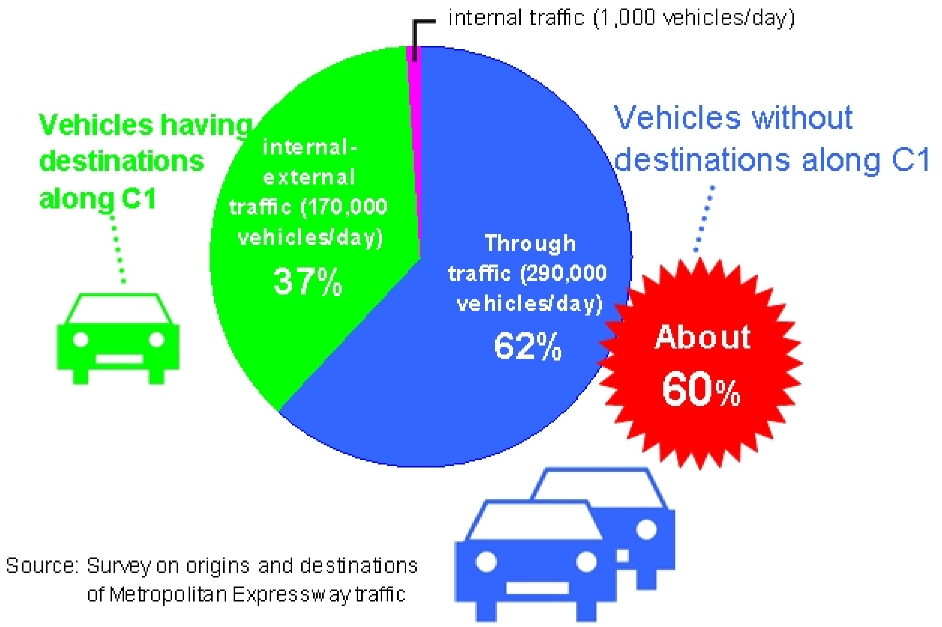



Figure 2. Traffic using the Inner Circular Route(C1)

With the opening of the Yamate Tunnel, through traffic can be diverted and dispersed, improving the overall flow of Metropolitan Expressway traffic and enabling efficient route selection (Figure 3).

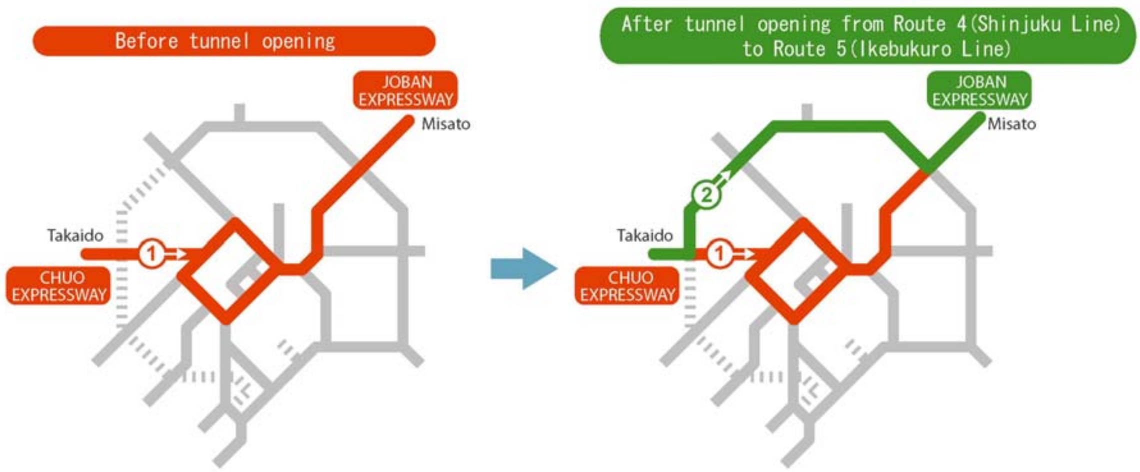



Figure 3. Efficient route selection

The diversion and dispersion of traffic is expected to reduce congestion on the Metropolitan Expressway and to provide important environmental and economic benefits.

## 3. CHANGES IN TRAFFIC AFTER OPENING OF THE YAMATE TUNNEL

### A. Changes in traffic volume

While growth in the overall volume of Metropolitan Expressway traffic has been stagnant, traffic volume of the Yamate Tunnel has steadily climbed . In November 2008, traffic volume averaged about 37,500 vehicles per day, or about 80% of the estimated volume (Figure 4).

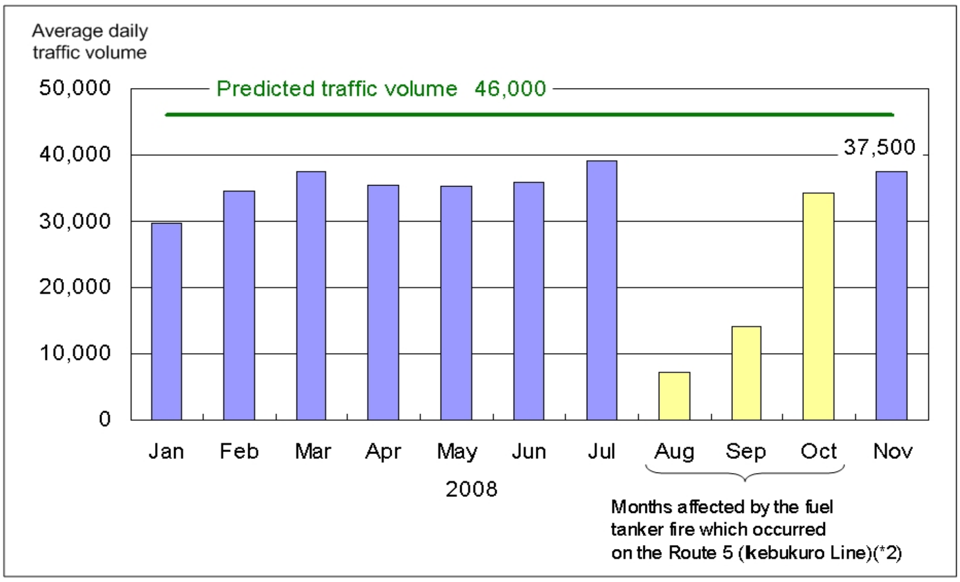



Figure 4. Trends in Yamate Tunnel traffic Volume

Comparing traffic volume before and after the Yamate Tunnel was opened, we see that traffic outside of C2 has increased, while traffic inside of C2 has decreased. This shift in traffic volume indicates that traffic has been diverted and dispersed (Figure 5).

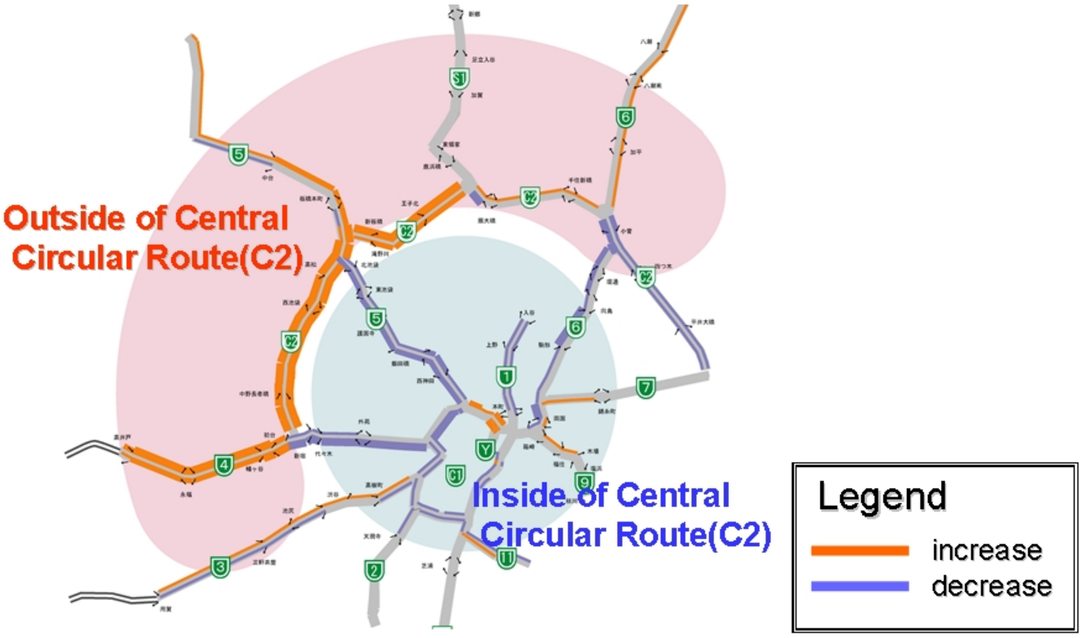



Figure 5. Traffic volume before and after the opening of the Yamate Tunnel

## B. Changes in congestion

### (1) Increase of "Smooth traffic flow" hours

As traffic has been diverted and dispersed, congestion has declined. There has been a particularly significant improvement on the Route 4(Shinjuku Route) in the direction toward central Tokyo. With improved travel speeds, smooth traffic flow (average speed of 40 km/h) now occurs on this portion for five additional hours per day, and the travel time has decreased at all times of day (Figure 6).

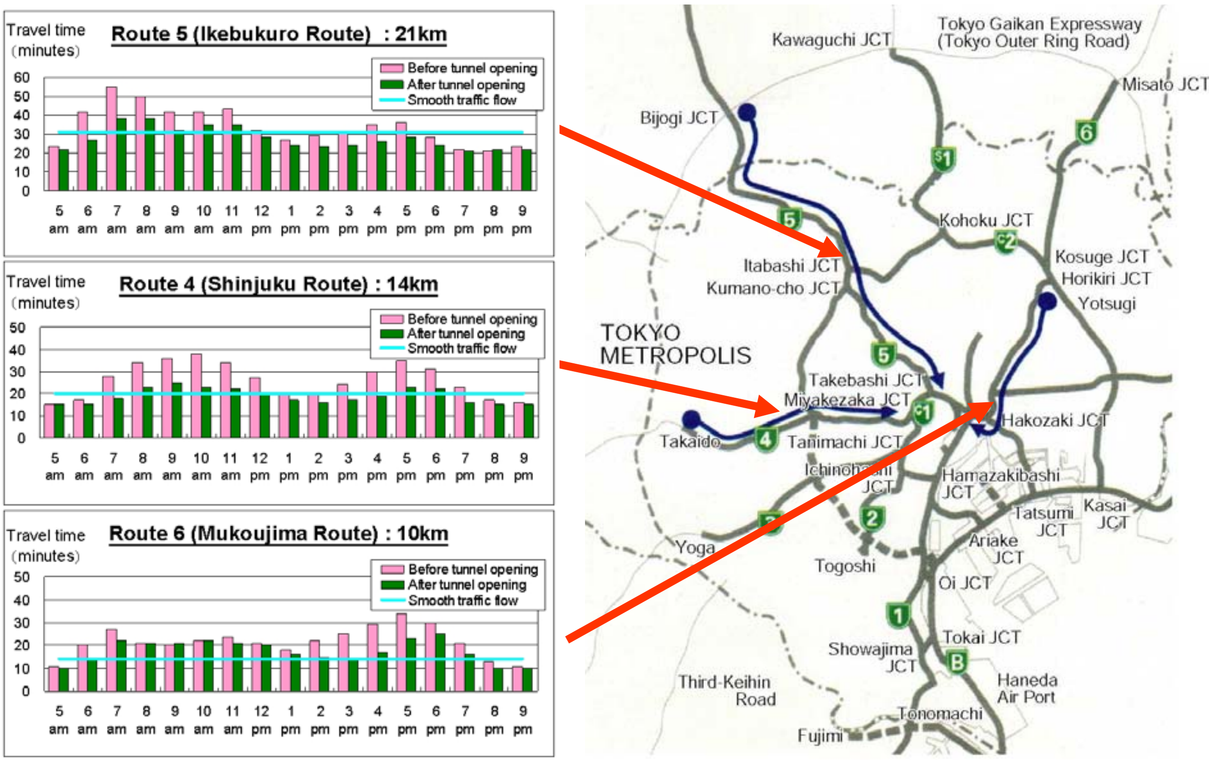



Figure 6. Changes in travel time, by time of day

### (2) Reduction of the irritation by congestion

From January to July, the length of congestion during the peak weekday hour (at 11 am) declined by an average of 26% compared to the same month of the previous year, before the tunnel was opened. This trend was again seen in November, after the effects of the tanker fire on the Route 5(Ikebukuro Route) had been eliminated (Figure 7). This has surely reduced the irritation of congestion. Figure 8 shows the changes in congestion by route during the peak weekday hour (at 11 am).

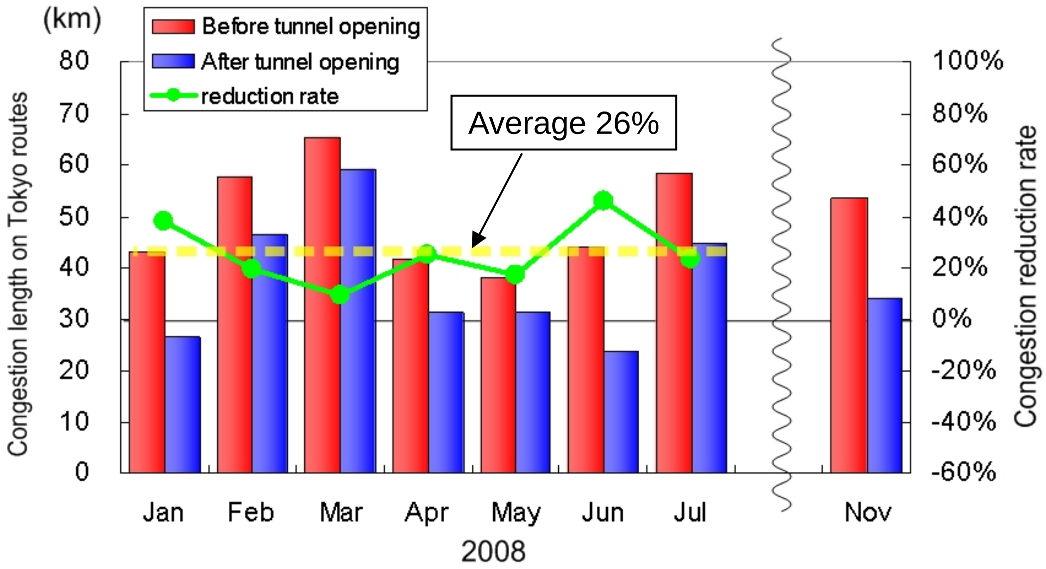



Figure 7.　Changes in congestion length during the peak weekday hour (at 11 am)

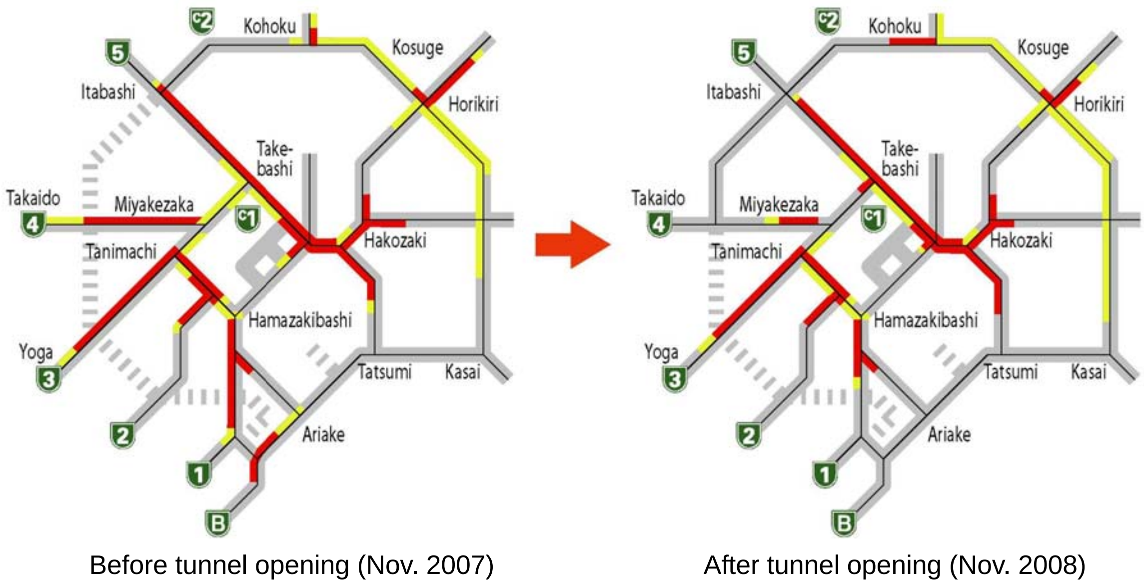



Figure 8.　Changes in congestion by route (November 2007 vs. November 2008)

**(3)　　Greater consistency in travel time**

The required travel time has decreased and become more consistent.　As shown in Figure 9, the average travel time from Takaido (Chuo Expressway) to Misato (Joban Expressway) was reduced by 14 minutes after the Yamate Tunnel was opened.　In addition, the maximum travel time was reduced by 33 minutes, resulting in a smaller difference in travel time depending on the time of day of departure.　This means that it will be easier for commuters and leisure travelers to estimate the required time and make accurate plans.

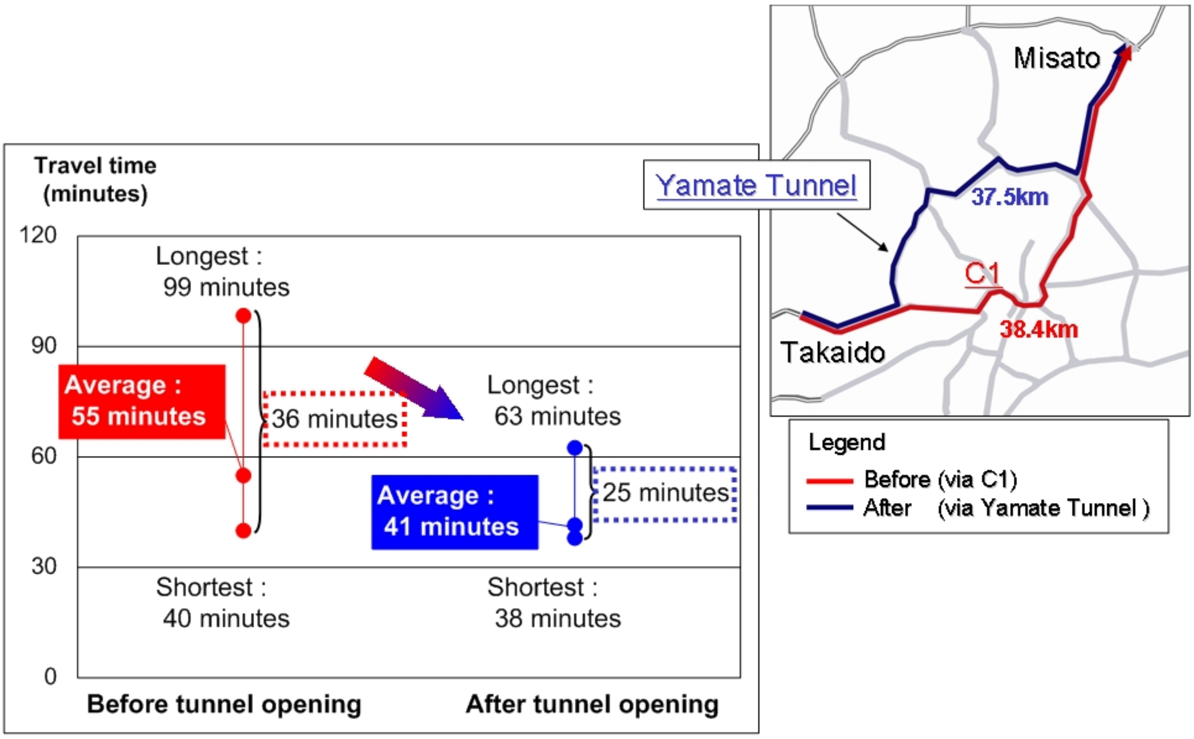



Figure 9. Changes in travel time (Takaido to Misato)

**(4) Fewer rear-end collisions in congested traffic**

With reduced congestion on the Route 4(Shinjuku Route) in the direction toward central Tokyo, there has been a 40% reduction in rear-end collisions occurring within a line of vehicles in congested traffic, indicating greater ease of driving (Figures 10 and 11).

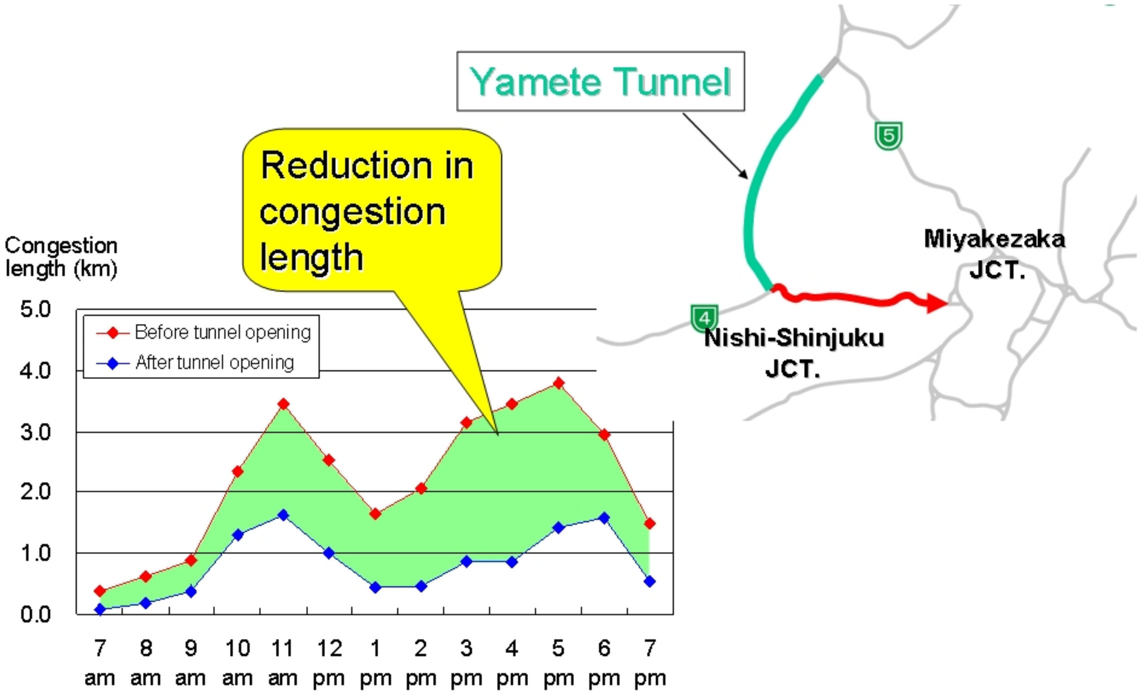



Figure 10. Congestion length by time of day

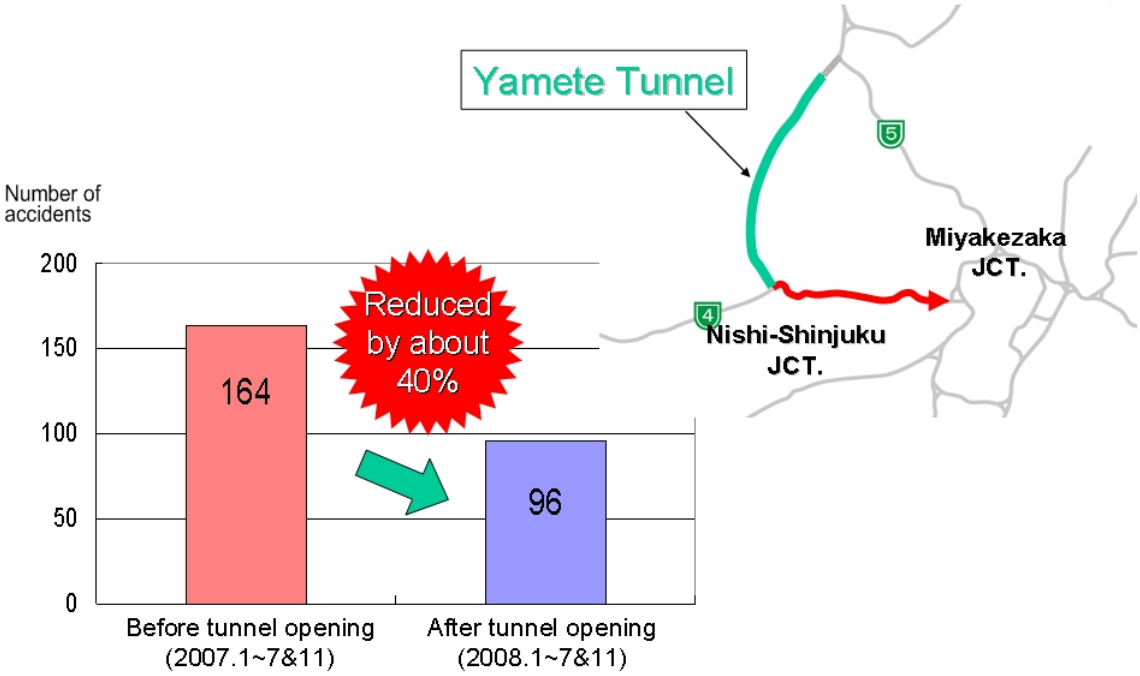



Figure 11. Traffic accidents before and after the Yamate Tunnel was opened

## 4. DRIVER RESPONSES FROM INTERNET SURVEY

### A. Survey summary

The survey was conducted to determine the usage situation of the Yamate Tunnel. Respondents were drivers from four prefectures (Tokyo, Saitama, Chiba, and Kanagawa) who had used the Metropolitan Expressway between January and May 2008, balanced by genders and age groups.

Survey period : May 8-10,2008

Sample size : 1,344 respondents

### B. Survey results

Figure 12 shows how drivers responded when asked what actual changes they have noticed as a result of the opening of the Yamate Tunnel. About 27% felt that the required travel time had decreased, and about 17% felt that the Metropolitan Expressway was less congested than before.

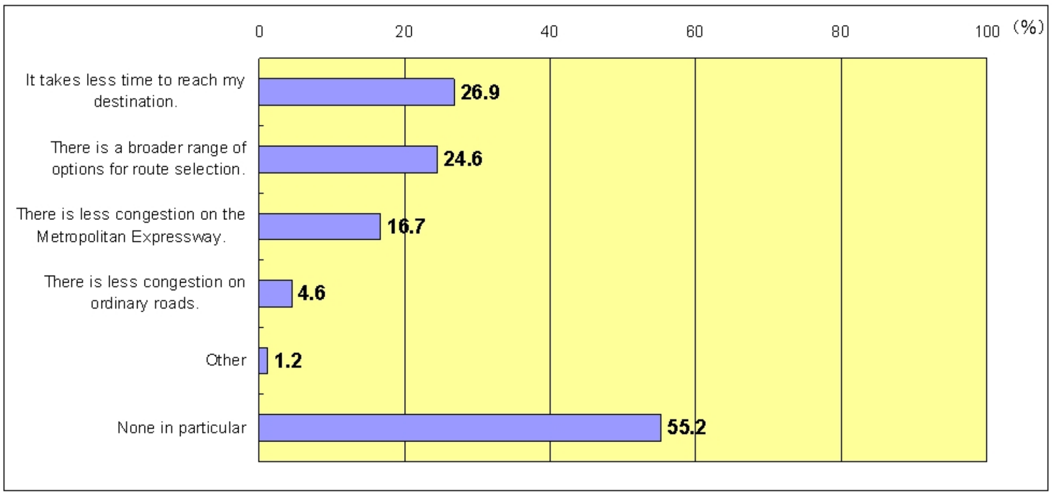



Figure 12.　Changes noticed as a result of the opening of the Yamate Tunnel

If a respondent stated that "There is less congestion on the Metropolitan Expressway" when answering the preceding question, they were then asked which routes have less congestion.　About half indicated that the Inner Circular Route (C1) and the Route 5(Ikebukuro Route) were less congested, and about 37% indicated that the Route 4(Shinjuku Route) was less congested.　This is in accordance with the figures from actual traffic measurements (Figure 13).

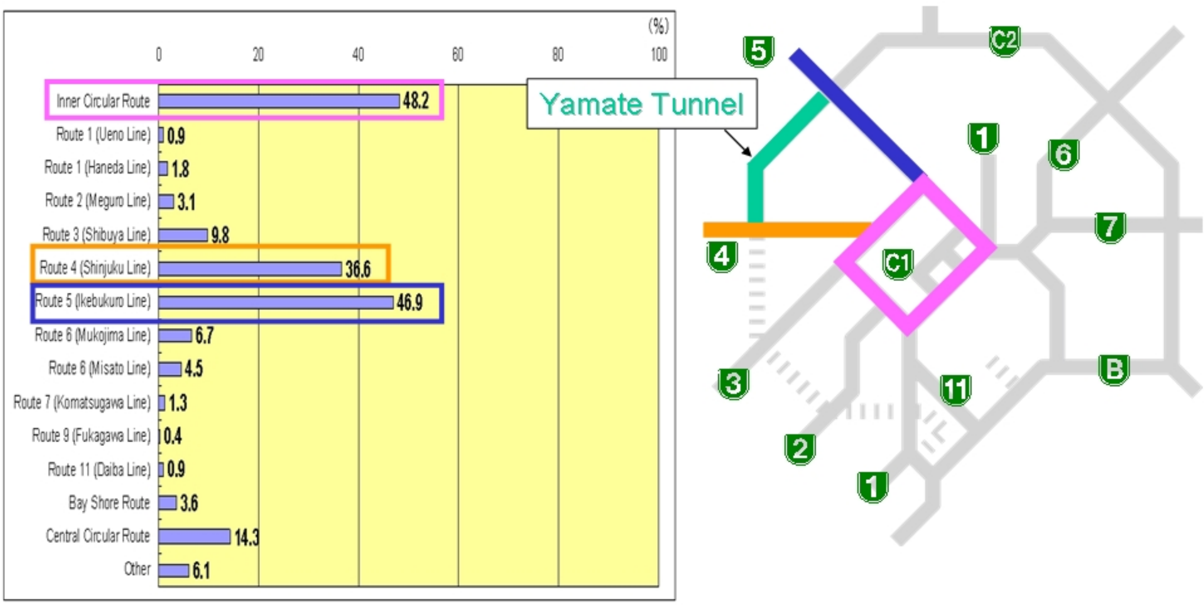



Figure 13.　Routes where drivers noticed lower congestion

Similar reactions were also obtained in an interview survey conducted in a parking area immediately after the tunnel was opened.　A truck driver stated, "There is a great time reduction compared to other routes, and this improves operating efficiency."　A company employee commented, "Traffic was really smooth and comfortable."　A female driver said, "It seems as though congestion is lower on other portions of the Metropolitan Expressway as well."

## 5. CONCLUSIONS

The Yamate Tunnel (linking the Route 4(Shinjuku Route) and the Route 5(Ikebukuro Route)) was opened nearly a year and a half ago. Since that time, circumstances such as soaring gasoline prices and a fuel tanker fire which occurred on Route 5 have made it difficult to directly measure the effects of tunnel opening. However, in the course of continuous data monitoring, it has become clear that the opening of the Yamate Tunnel has resulted in significant benefits for the Metropolitan Expressway.

In the future, we plan to verify whether the proportion of through traffic using the Inner Circular Route (C1) has been reduced from the 60% level, as mentioned at the beginning of this report, and to analyze the environmental benefits and economic effects of the tunnel.

The remaining 4.3-kilometer section of the Yamate Tunnel, extending from the Route 3 (Shibuya Route) to the Route 4 (Shinjuku Route), is scheduled to be opened in March, 2010. Next, the Central Circular Shinagawa Route, which is the last remaining section of the Central Circular Route (C2), is scheduled to be opened in FY 2013.

It is anticipated that interest will continue to grow regarding the effects of improvements on the Central Circular Route, and we are planning to engage in further studies to verify these effects.

## REFERENCES


1 Report of the 25th survey on origins and destinations of Metropolitan Expressway traffic

2 For more information concerning the fuel tanker fire on the Route 5(Ikebukuro Route): http://www.shutoko.jp/route5/index.html

# The traffic accident reduction measures by climbing lane in Meishin Expressway

**Koichi MATSUMOTO**
*Ibaraki Operation Office, Kansai Head Office,*
*West Nippon Expressway Company Limited*
*4-10-1 Kamihozumi Ibarakishi, Osaka, 567-0036, JAPAN*
*k.matsumoto.aa@w-nexco.co.jp*

**Aimi MOROOKA**
*Ibaraki Operation Office, Kansai Head Office,*
*West Nippon Expressway Company Limited*

**Sihyeong AN**
*West Nippon Expressway Engineering Kansai Co., Ltd.*

**ABSTRACT**

The 6-lanes section between Oyamazaki IC and Ibaraki IC in Meishin Expressway is an eminent heavy traffic section in Japan, traffic volume is 126,000 cars per a day.

Recently, in the neighboring of Takatsuki-Bus-Stop on the side for Osaka in this section, the occurrence of traffic accident is remarkable, and it is necessary to take traffic accident reduction measures immediately.

We analyzed the cause of the traffic accident in Takatsuki-Bus-Stop area.

In there, a lot of congestion due to road structure called sag occurs, and many of traffic accidents are rear-end collision in congestion queue or at the end of congestion queue.

Particularly, many of the rear-end collision occur in passing lane.

By our study, it was become clear that occurrence of the traffic accident have much to do with congestion.

We proposed that it is effective accident prevention measures to increase the road capacity by installing climbing lane.

## 1. INTRODUCTION

The Meishin Expressway opened in July 1963 as the first expressway in Japan and has marked the 46th anniversary of the opening this year. Since the opening, it has greatly contributed to the Japanese economy development as the main transportation artery linking eastern and western Japan. The annual mean daily traffic within the six-lane section between the Oyamazaki IC and the Ibaraki IC exceeded 126,000 vehicles in 2008, and thereby this section has become one of the most heavy-trafficked

sections of expressways in Japan.

An area near the Takatsuki BS located on the down lane (the side for Osaka) in the relevant section has a sag road construction (of -4.6% to +4.0%), where is a common site of traffic jams and accidents. Due to an increase in traffic in the relevant section after the Shin-Meishin Expressway was put in service in February 2008, the number of traffic jams and that of traffic accidents are both further increasing. Under these circumstances, it has been anticipated to implement immediate measures for the prevention of traffic accidents.

We hereby make a report on the measures for the prevention of traffic accidents that we implemented by installing a climbing lane.

Photo-1　Down lane at Takatsuki BS

Photo-2　Traffic jam at Takatsuki BS

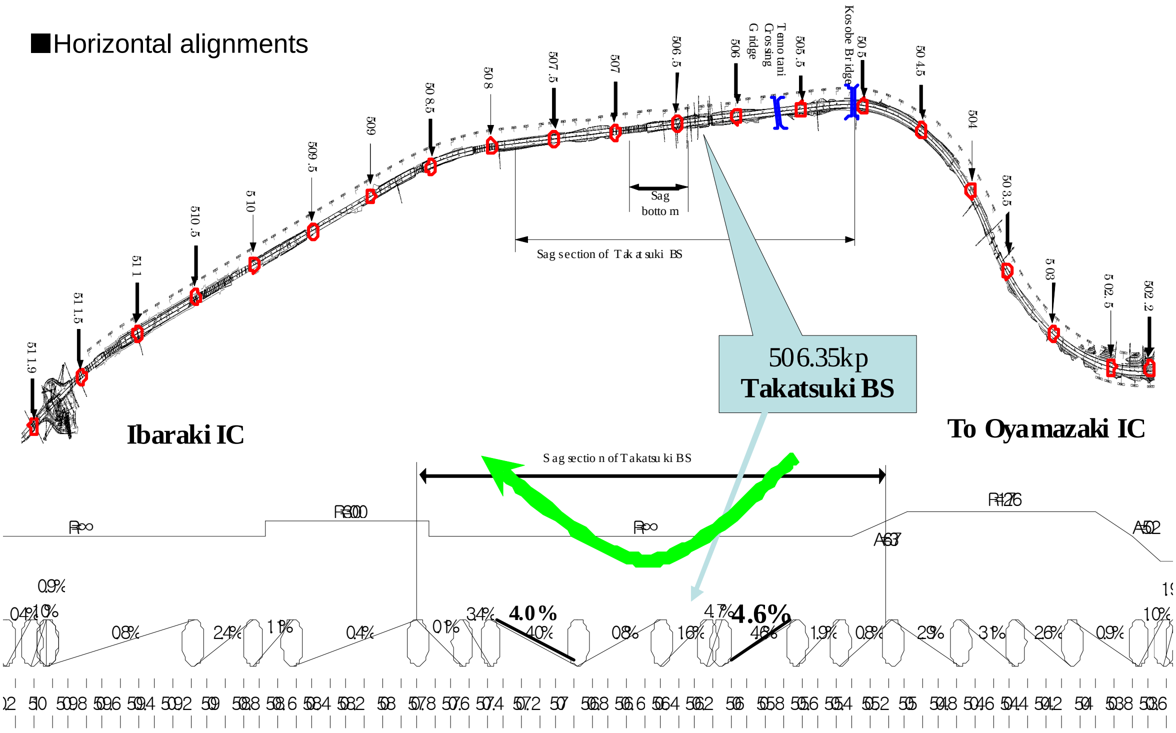



Fig-1　Road alignments around Takatsuki BS

## 2. TRAFFIC SITUATION IN THE NEIGHBORING OF TAKATSUKI-BUS-STOP

### 2-1. Traffic jam occurrence situation

Regarding the situation of traffic jams that occurred from the area on the down lane (the side for Osaka) of the Meishin Expressway in the neighboring of Takatsuki-Bus-Stop, such traffic jams occurred 68 times in 2004, while the number of traffic jams sharply increased to 133 times in 2008. Furthermore, compared to the number of traffic jams that occurred on the up line (the side for Nagoya), traffic jams occurred on the down line approximately seven times as many as those on the up line on an average during the period of 2004 to 2008. (See Table-1)

Table-1 Traffic situation around Takatsuki BS

(1) Annual mean daily traffic (Vehicles/day)

| | Up lane | Down lane | Total |
|---|---|---|---|
| 2004 | 59,000 | 59,065 | 118,065 |
| 2005 | 61,198 | 61,151 | 122,349 |
| 2006 | 62,612 | 62,320 | 124,932 |
| 2007 | 62,704 | 62,450 | 125,154 |
| 2008 | 63,272 | 63,527 | 126,799 |

| | |
|---|---|
| Mar. 30, '03 | Keiji ByPass opened between Ogura IC to Kumiyama JCT Second Keihan Expressway opened between Oguraike IC and Hirakata-higashi IC |
| Aug. 8, '03 | The entire portion of the Keiji ByPass opened. (Completed the root twinned with Meishin Expressway) |
| Feb. 23, '08 | Shin-Meishin Expressway opened Kusatsutanakami IC and Kameyama JCT. |

*Source for 2008: Quick estimation from Traffic Counter Data

(2) Number of traffic jams (Jams/year)

| | Up lane | Down lane | Total | (Down / Up) |
|---|---|---|---|---|
| 2004 | 10 | 68 | 78 | * 7.22 |
| 2005 | 15 | 97 | 122 | |
| 2006 | 20 | 110 | 130 | |
| 2007 | 11 | 119 | 130 | |
| 2008 | 17 | 133 | 150 | |

*Mean value of the period of 2004 to 2008

*Traffic jams caused due to heavy inbound traffic

(3) Number of accidents (Accidents/year)

| | Up lane | Down lane | Total | (Down / Up) |
|---|---|---|---|---|
| 2004 | 4 | 14 | 18 | * 3.45 |
| 2005 | 3 | 20 | 23 | |
| 2006 | 15 | 58 | 73 | |
| 2007 | 14 | 48 | 62 | |
| 2008 | 20 | 53 | 73 | |

*Mean value of the period of 2004 to 2008

*Accidents caused in the sag section of the Takatsuki BS

### 2-2. Traffic accident occurrence situation

Regarding the situation of traffic accidents caused in the neighboring of Takatsuki-Bus-Stop on the down lane of the Meishin Expressway, we analyzed 30 traffic accidents caused in the section between 505.5kp and 506.7kp during the year 2007. These accidents include three characteristics: Rear-end accidents acount for approximately 80% of the total, accidents related to traffic jams acount for approximately 70% of the total, and most of the accidents were caused on the passing lane. (See Fig-2)

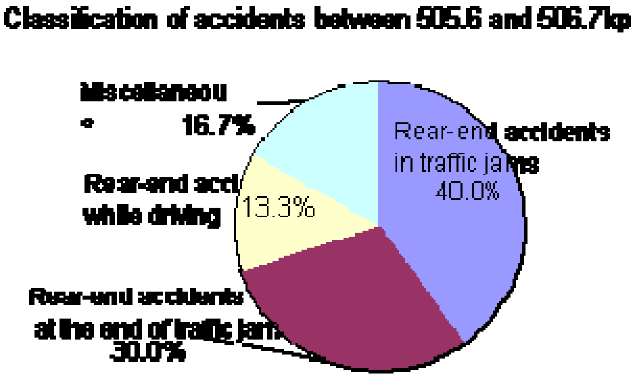



Fig-2　Classification of traffic accidents by pattern

Photo-3　Multiple rear-end accident

Regarding the situation of traffic accidents caused, rear-end accidents caused in traffic jams were relatively minor accidents. However, rear-end accidents caused at the end of traffic jam are more likely to multiple rear-end accidents involving a number of vehicles and highly likely to multiple fatal rear-end accidents occasionally. If a multiple rear-end accident occurs, the expressway will be closed to vehicles for an extended period of time, involving many customers in the closure and also raising concerns that another traffic jam starting from the accident site and secondary traffic accidents are caused. (See Photo-3)

Consequently, it has been required to implement urgent measures for the prevention of traffic accidents, that is, those for the prevention of traffic jams.

### 3. MEASURES FOR PREVENTION OF TRAFFIC ACCIDENTS BY INSTALLING CLIMBING LANE

Since traffic accidents related to traffic jams acount for 70% of the total, preventing traffic jams from being caused by sag construction is considered effective in preventing traffic accidents. The best preventive measure against traffic jams is to expand the traffic capacity of the relevant section, that is, to increase the number of traffic lanes from the current three lanes to four lanes. However, since adding just one traffic lane requires substantial reconstruction work, it is considered impractical as urgent measure. Under these circumstances, we made a study on the way to prevent traffic jams by installing a climbing lane as a measure that can be implemented without any change to the current total width of road.

#### 3-1. Comparison of traffic by time zone (on up and down lanes)

Even though a comparison of peak hour traffic between the up lane and the down lane indicates approximately the same level, that is, a little less than 4,500 vehicles per hour, there are the fewer number of traffic jams and accidents on the up lane. The supposed reason is that a climbing lane has been built on the up lane since the opening of the expressway, and that the climbing lane has contributed to preventing traffic jams.

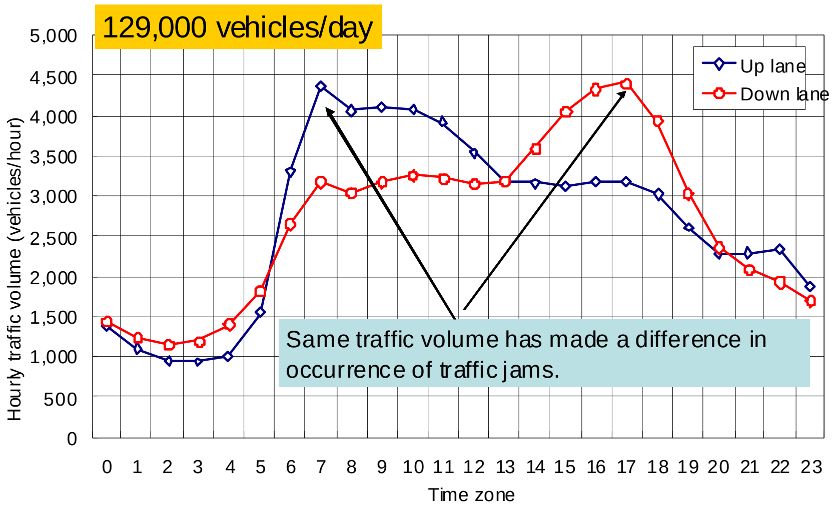



Fig.-3　Changes in hourly traffic volume between the up lane and the down lane

Photo-4　　Up lane at Takatsuki BS

### 3-2. Analysis of climbing lane and its utilization rate

In order to identify differences in traffic situations and traffic jam occurrence situations between before and after building a climbing lane, we analyzed the relationship between traffic and climbing lane utilization rate.

A fixed traffic measurement system is installed at 506.1kp on the Meishin Expressway. However, since the system is located approximately 100 meters downstream from the starting point of the climbing lane on the up lane and further the climbing lane is just starting, data collected by this system remains not available to make a definite judgment on differences in lane utilization rates between the up lane and the down lane.

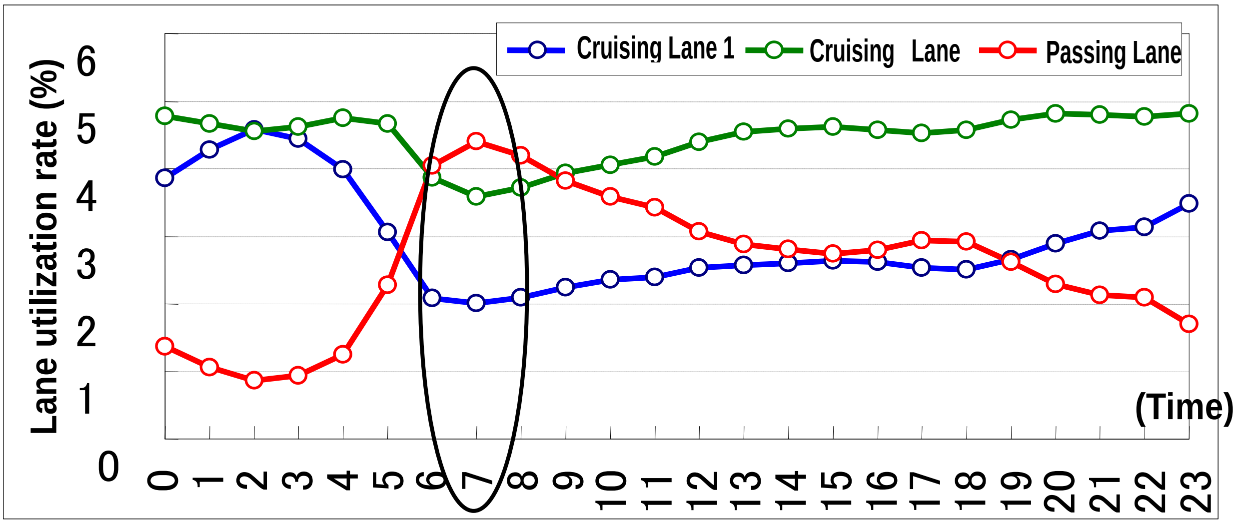
To identify changes in the lane utilization rates with the provision of the climbing lane, we analyzed data collected by fixed traffic measurement systems installed at two points located upstream from Nishinomiya-Najio SA on the down lane of the Chugoku Expressway (i.e., 21.05kp and 23.12kp) where a climbing lane is built in a six-lane section similar to the relevant section and no traffic jams are observed in such section.

Regarding the lane utilization rates at seven o'clock, at which peak hour traffic reaches approximately

4,500 at 21.05kp (located on the upstream from the starting point of the climbing lane with a longitudinal road alignment of 1.7%) and 23.12kp (located 400 meters downstream from the starting point of the climbing lane with a longitudinal road alignment of 4.0%), the utilization rate of the passing lane shows a 3%, declining from 44% to 41%, while that of the climbing lane come to 6%.

As a result, we think that, in the section with the climbing lane built, traffic jams on the passing lane has been successfully prevented without eventually causing traffic on the passing lane to exceed the traffic capacity by reducing traffic that had been unevenly distributed to the passing lane by 200 vehicles/hour (in other words, by reducing the lane utilization rate by approximately 5%) to make such reduced traffic change sequentially to the cruising lane 2, cruising lane 1, and then climbing lane.

21.05kp (located on the upstream from the starting point of the climbing lane)




23.12kp (located 400 meters downstream from the starting point of the climbing lane)

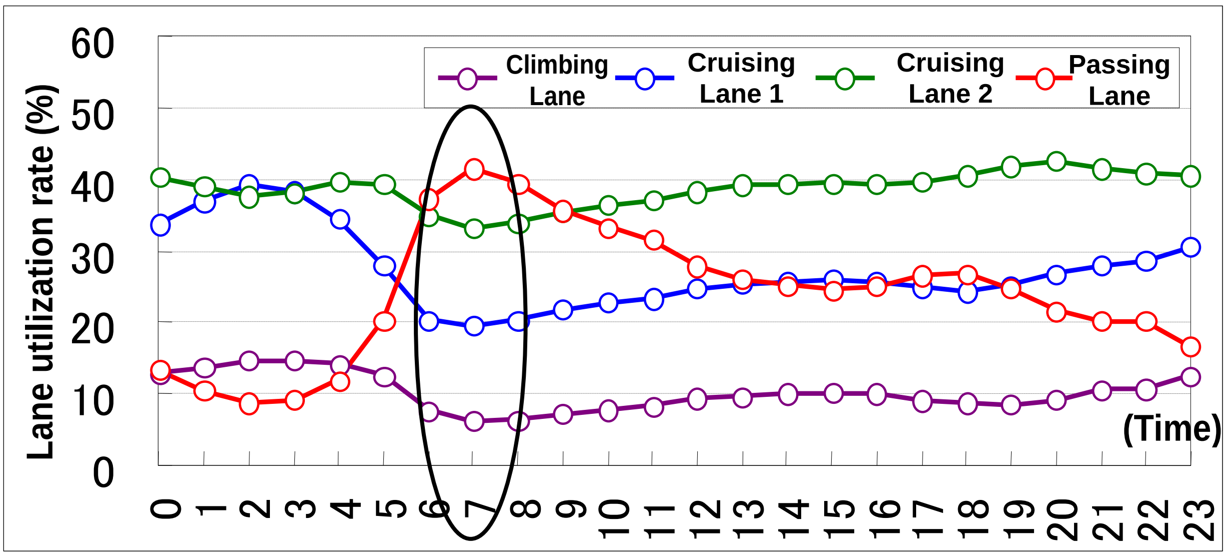


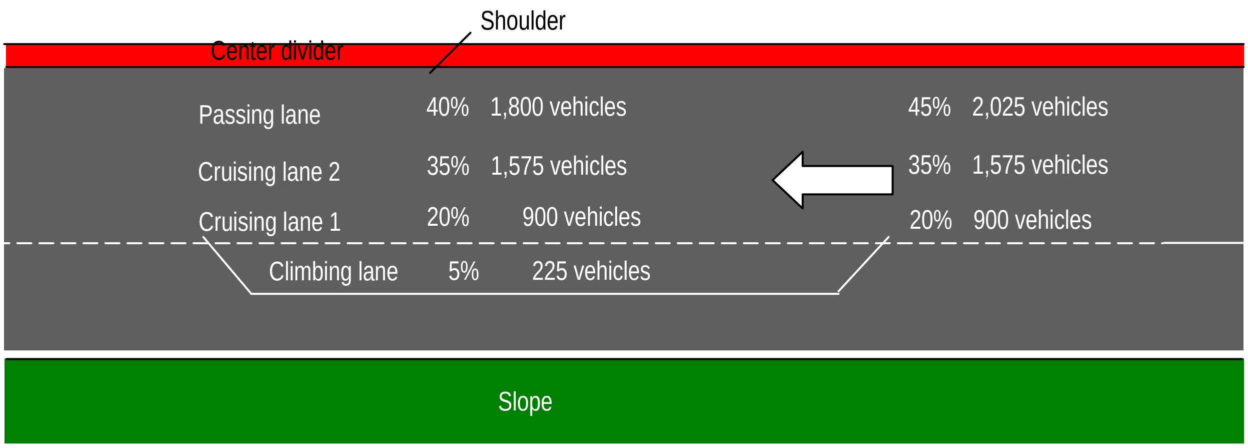

Fig.-4　Changes in lane utilization rates by time zone

Fig. 5 shows changes in lane utilization rates and traffic expected with provision of climbing lane.

Assuming that peak hour traffic reaches 4,500 vehicles at the sag area in the relevant section when a

traffic jam occurs, we expect that such traffic jam within the section with climbing lane built can be prevented by reducing the traffic of 2,025 vehicles (the utilization rate of 45%) unevenly distributed on the passing lane to the traffic of 1,800 vehicles (the utilization rate of 40%, resulting in just an approximately 5% decline).

Consequently, we expect that reduction of traffic jams will achieve a 50% or more reduction of the current number of traffic accidents.




Peak hour traffic: 4,500 vehicles / hour

Fig.-5 Expected changes in lane utilization rates and traffic

## 4. STUDY ON STRUCTURE OF ROAD WIDTH

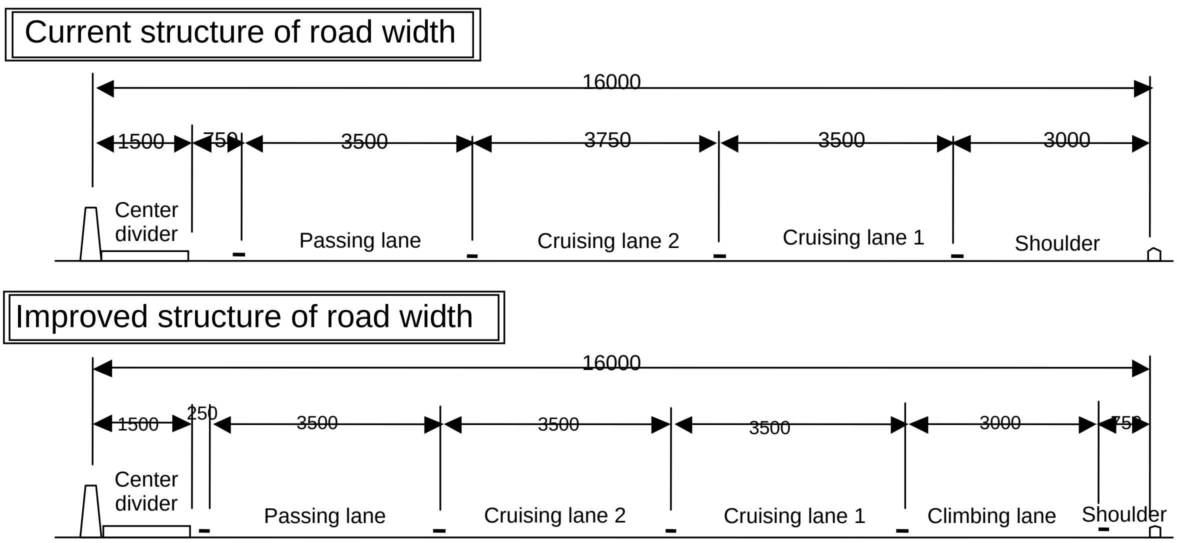



Fig.-6 Comparison of structures of road width

Since the relevant section of the Meishin Expressway falls under the road category of Type 1, Class 2 of the Road Standards, it is impossible to build climbing lane within the current full road width with the structure of road width meeting these Road Standards

However, we could installed the climbing lane without making any changes to the current full road

width by taking the structure of road width shown in Fig.-6 with the application of special values prescribed under the Road Structure Ordinance.

## 5. CONCLUSION

Since 70% of accidents caused in the relevant section were related to traffic jams, we made a study on installing a climbing lane for the sag area (the neighboring of Takatsuki-Bus-Stop ) on the down lane of the Meishin Expressway to prevent traffic jams caused there by the sag construction as a measure for the prevention of traffic accidents.

For installing the climbing lane, we found out that it could be installed by applying the special values prescribed under the Road Structure Ordinance without making any changes to the current road width.

In addition, we expect that installing the climbing lane will be effective in eliminating traffic jams and further achieving a significant reduction in accidents caused during traffic jams.

Since the climbing lane as a measure for the prevention of traffic accidents at the sag area can be installed just by changing the division lines and shifting the lanes, this measure is considered implementable in a short period of time, most effective, and most economical as an urgent measure.

# Advanced Road Data for Transportation and Traffic Engineering

**Brendan Marsh, Main Roads Western Australia**

## Abstract

Road engineering is heavily dependent upon good data and predictive modeling of traffic volumes and conditions. Recent rapid technology advances has made it possible to gather real time datasets at the detailed level based upon real time Vehicle Detection Stations and process vast quanta of vehicle by vehicle data. However, there are many challenges with building historical and real time systems that can effectively manage the data volumes and deliver advanced information that enhances road engineering planning and design. Main Roads Western Australia (MRWA) has taken up the challenge with the cutting edge Network Intelligence Project and a key outcome already achieved is the development of an innovative "Data Cube". As a result, MRWA has developed capability to report upon the newly formed National Performance Indicators (that are based upon the detailed real time traffic data collected throughout the year) for freeways through an automated system. MRWA can also report upon a wide range of traffic engineering, social, economic and environmental parameters (such as greenhouse gas emissions), through an estimation process that considers vehicle by vehicle parameters reported by the established Vehicle Detection Stations.

### 1. BACKGROUND

Main Roads Western Australia (MRWA) is the State road (transport) agency, with responsibility for some 17 800km of major roads, all traffic signals and all regulatory road marking and signage in Western Australia. The geographical distribution of MRWA's assets exceeds all the other road transport agencies and MRWA is considered to be one of the key stakeholders in Intelligent Transport Systems (ITS).

MRWA developed its ITS strategy for 2005 and 2010 with a key objective being:

> 'By 2010 Main Roads will be recognized as a leading Australian road agency in achieving the benefits of Intelligent Transport Systems'[1].

The Strategy points to the following ITS Objectives:

1. Timely and accurate information to road users and managers;
2. Effective control of road use;
3. Improved road safety, access and compliance;
4. ITS capability, resources and awareness of developments; and
5. Minimise risk to Government.

Advanced road data is relevant to the achievement of all Strategy objectives, resulting in the initiation of the *Network Intelligence Project* (NIP) and a Network Intelligence Technical Reference Group (NITRG) to oversee and contribute to the project.

The purpose of the NIP is to provide the data and information foundation for current and future ITS systems within an efficient and robust architecture that can be supported in the long term. This paper describes the NIP, documents major achievements to date and summarises future planning.

[1] Intelligent Transport Systems Strategy 2005 – 2010, Main Roads Western Australia, December 2004.

## 2. MOTORWAY/FREEWAY VEHICLE DETECTION STATION DATA

Prior to the ITS Strategy, Main Roads had few Vehicle Detection Stations located on its freeway (motorway) network and the only source of operational information was through Closed Circuit Television (CCTV) imagery. By contrast, datasets did exist for the arterial road network through the Sydney Coordinated Adaptive Traffic Signal (SCATS) system. Accordingly, a program to install foundation Vehicle Detection Station (VDS) infrastructure along the freeway network was initiated.

These circumstances affected the priorities for the NIP as work was required to define the data set to be delivered and to establish systems to collect and manage the data.

### 2.1 The Raw Data

While many road authorities collect aggregated data from their motorways, generally ranging from 20 second (e.g. Victoria) to 2 minute aggregations, interview with practitioners revealed a view that this was principally to reduce computer processing and data storage requirements.

The few sites we already had in place were sending vehicle by vehicle records, including attributes such as:

- VDS Number;
- Record Number;
- Date and Time;
- Traffic Lane;
- Vehicle Direction;
- Vehicle Speed;
- Vehicle Length;
- Headway; and
- Gap.

Detailed assessment of the feasibility to retain the existing data format for the pending VDS sites was undertaken and it was found that data storage and processing costs had fallen sufficiently for this to be considered a suitable alternative. In addition, the NITRG agreed that aggregation of the data would limit future flexibility in operations, analysis and reporting. Accordingly, vehicle by vehicle data records were agreed and implemented for all VDS.

### 2.2 Data Cube – The Future of Historical Analysis

Investigation was commissioned to consider the data analysis needs of Main Roads and the most suitable methodology given the “Raw Data” decision noted above and emerging software technology solutions. It was found that ultimately a number of

application specific options would be required, however, for fast access to detailed data views to meet most of Main Roads requirements, a Data Cube would provide the best solution for the first project iteration.

A data cube has some similarity to a traditional data base that contains data aggregations such as 1 minute data averages / totals. The traditional data base contains two “dimensions” involving space (the location of VDS and traffic lanes) and time; by comparison, a data cube has more dimensions. Within each cell formed by the dimensions are the Measures, which is identical to the data within the cells of a traditional database.

The Main Roads Data Cube is allowing a wide range of analysis to be undertaken ranging from detailed views of network performance to National Performance Indicator reporting.

#### 2.2.1 Main Roads’ Data Cube Dimensions

The role of the Dimensions in a data cube is similar to a library catalogue; they structure the data to increase the efficiency at which data may be retrieved.

Main Roads’ data cube presently has five dimensions:

1. Space;
2. Time;
3. Speed;
4. Vehicle Length; and
5. Traffic Density.

The Space dimension is very similar to a traditional database and contains aggregation options such as traffic lanes, freeway sections, freeway directions, freeway names and the freeway network.

The Time dimension is very similar to a traditional database and contains aggregation options such as minutes, peak period, working day, school holiday, day of week, day of month, quarter and annual.

The Speed dimension sorts data records into speed range place holders. This enables calculations and aggregations to be completed that are only associated with vehicles travelling within certain speed ranges. For example, the speed enforcement agency may wish to know where and when they should be focusing their speed enforcement

attention; analysis that is enabled by this dimension. A vehicle by vehicle database would also enable this analysis, however, this is not enabled with traditional aggregated freeway data.

The Vehicle Length dimension is fundamentally length based vehicle classification and like the speed dimension, it allows measures to be viewed for selected vehicle length categories. For example, a freight traffic restriction was imposed on a section of Leach Highway, seeking re-routing of more freight traffic onto an alternative route on Kwinana Freeway. The Vehicle Length dimension allows data for the restricted vehicles to be quickly considered and compared.

The Traffic Density dimension relates to the Highway Capacity Manual and Level of Service, which is fundamentally defined by density. It considers the number of vehicles experiencing different traffic densities based upon vehicle separation (headway) and vehicle speed.

With these dimensions in place, a data analysis / search can quickly focus upon the data cells that are relevant to the search criteria compared with requiring a search through all data within the specified time and space range. Further, the data cube pre-calculates key data aggregations, significantly reducing the time associated with data extraction.

### 2.2.2 Main Roads' Data Cube Measures

Main Roads' Data Cube currently contains 33 different measures, such as speed, volume, density, occupancy, estimated $CO_2$ emissions, 'avoidable' $CO_2$ emissions, etc. Each measure is calculated from the fundamental line of vehicle data as data is entered into the cube. For example, traffic density is calculated by traffic volume divided by vehicle speed. In addition, the "volume" for a single vehicle is based upon the inverse of the headway.

The measures are calculated for every vehicle record and then aggregated into the relevant cell of the Data Cube. For example, if a vehicle data line was processed and found to be travelling at 82.5km/h, experiencing traffic density of 26.2 veh/km, of length 18.2m at the location of VDS 10 and recorded at 9:01:30am 19 February 2009, then it's measures would be recorded within the cube cell Speed: 82-83km/h, Level of Service D, Vehicle Class 2, Mitchell Freeway 0 - 0.5km and 9:01 – 9:02am 19 February 2009.

## 2.3 Advanced Data Sets and Detailed Historical Analysis

The Main Roads' Data Cube allows a wide range of detailed data views, supporting detailed Traffic Engineering analysis of freeway performance and planning of future

projects and Intelligent Transport Systems. In addition to enabling traditional speed, volume, occupancy and density data views, the consideration of advanced information and datasets has been enabled and this is moving towards a sustainability framework.

Analysis undertaken for Kwinana Freeway considering the “direct costs” of congestion during the morning peak period and the Main Roads Data Cube enabled a cost profile (see Figure 1) and heat map (see Figure 2) to be prepared. The “direct costs” of congestion is a conservative calculation that is based upon the extra travel time, excessive travel reliability time allowances, fuel consumption and environmental emissions (e.g. $CO_2$, CO, $NO_x$, VOC and Particulates) that arises when vehicle speeds fall beneath 60km/h. The choice of 60km/h was based upon optimums for freeway productivity, fuel consumption and emissions coinciding with approximately 70km/h average freeway speed and the view that with freeway management, almost all vehicles should be travelling above 60km/h if laminar flow is maintained.

Similar plots can be prepared outlining the net costs of transport within these areas.

The model is including economic and environmental costs associated with transport, noting also that the travel time element does have social implications for commuters. Therefore, the analysis that has been enabled is heading in the direction of a Sustainability view of transport operations.

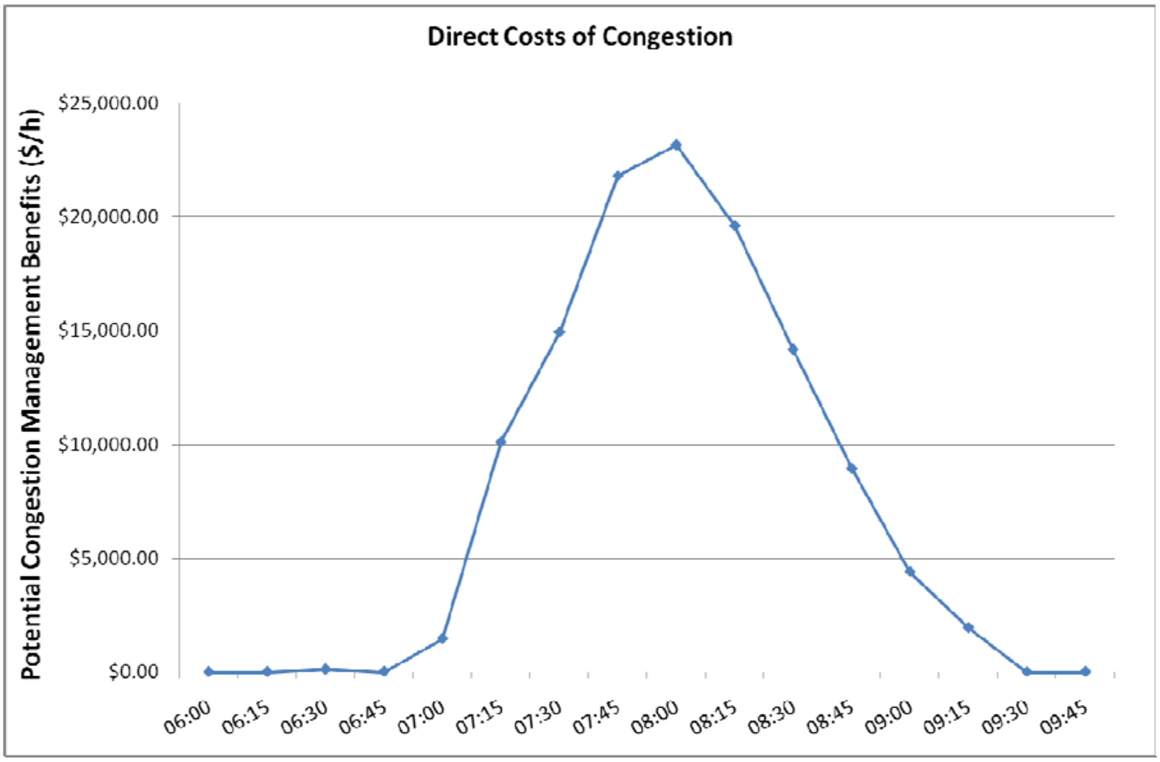


Figure 1: Direct costs of congestion Kwinana Freeway north bound, Wednesday 19 November 2008.

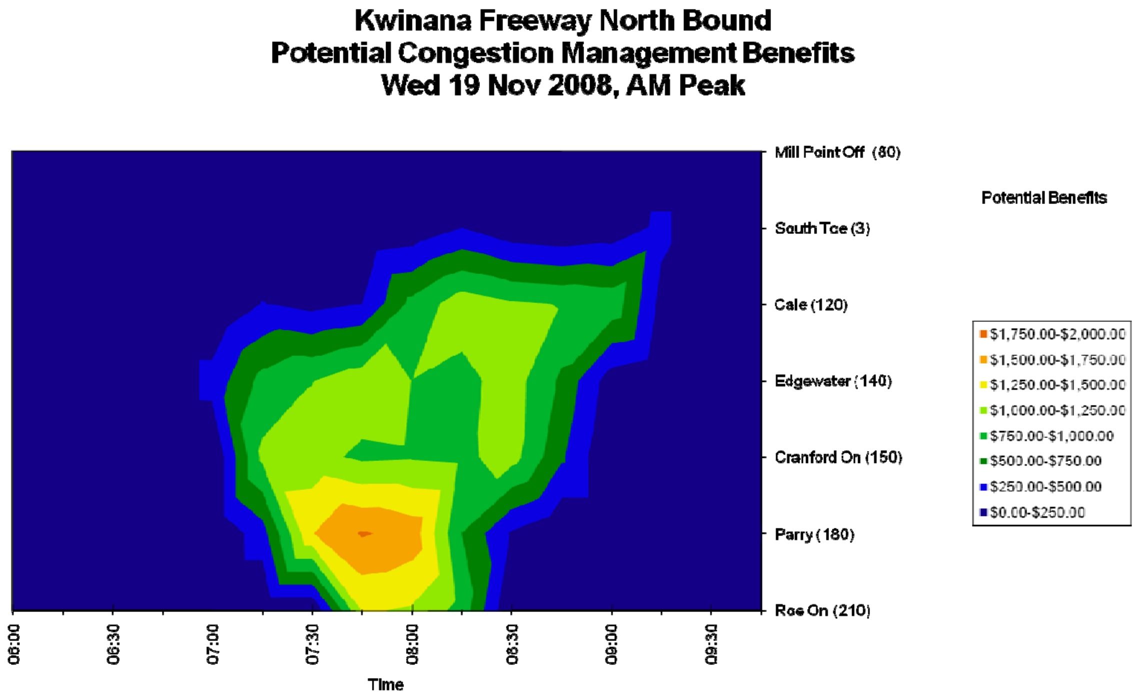



Figure 2: Potential benefits of applying advanced ITS to prevent productivity collapse based upon time and location.

The current Main Roads Data Cube is actually a prototype for the future, based upon the current freeway detailed traffic data set. Future directions include:

- Improved data screening and validation procedures;
- The consideration of other transport cost measures associated with sustainability such as noise, access, and equity;
- The consideration of transport cost measures associated with resilience; and
- The expansion of the cube to provide a network view including traffic signal data sets.

### 2.4 National Performance Indicators and Other Performance Reporting

Austroads Project NS1207 developed 'operational performance indicators suitable for the automated measurement of network performance'[2]. Based upon the Main Roads Data Cube, an application has been developed to provide reporting in accordance with the National Performance Indicator (NPI) methodology for Perth's freeways.

[2] R. Troutbeck, M. Su, J. Luk, Austroads Research Report, National Performance Indicators for Network Operations, AP-R305-07, Austroads, September 2007.

There are four NPI reports for freeways:

- Efficiency (Average Travel Speed);
- Efficiency (The percentage of freeway operating within levels of variation from the posted speed limit);
- Productivity (Percentage of network operating within different productivity bands); and
- Reliability (Percentage of network operating with differing degrees of travel speed fluctuations from day to day through the reporting period).

The reports consider working day data during the selected peak periods and aggregate the calculation outcome across the time period in time and space. For the Efficiency (Average Travel Speed) indicator, this results in a single number representing the average travel speed – see Figure 3. The outcome for the other indicators represents the percentage of the freeway performing within each performance band both in time and space – see Figures 4, 5 and 6.

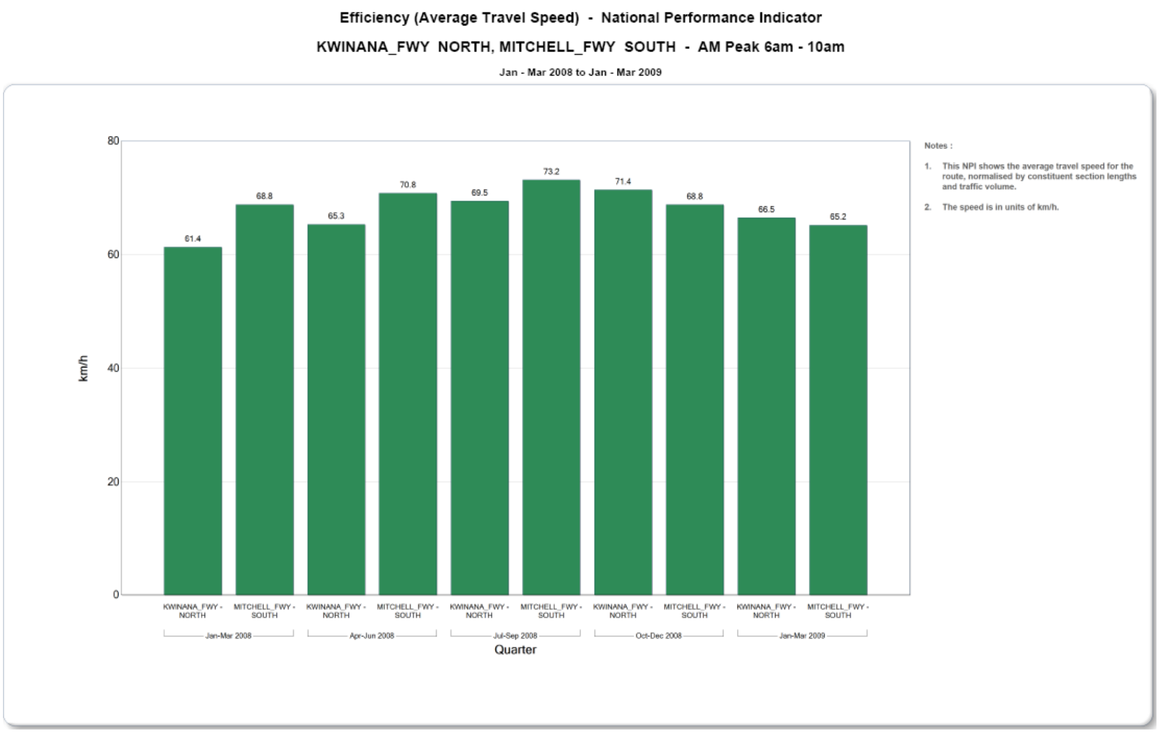



Figure 3: National Performance Indicator report comparing the Efficiency (Average Travel Speed) of Kwinana Freeway (north bound) and Mitchell Freeway (south bound) during the morning peak period between 6am and 10am.

**Efficiency (Variation from Posted Speed Limit) - National Performance Indicator**

**KWINANA_FWY NORTH, MITCHELL_FWY SOUTH - AM Peak 6am - 10am**

**Jan - Mar 2008 to Jan - Mar 2009**

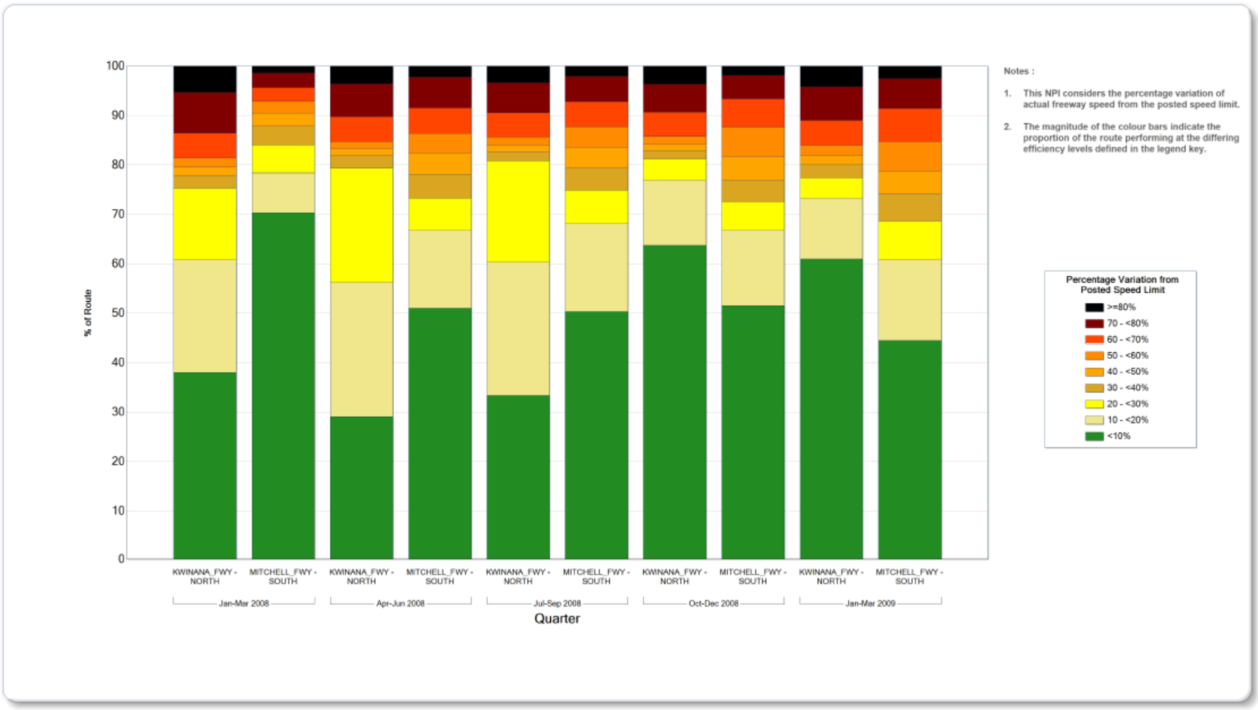



Figure 4: National Performance Indicator report comparing the Efficiency (Variation from Posted Speed Limit) of Kwinana Freeway (north bound) and Mitchell Freeway (south bound) during the morning peak period between 6am and 10am.

**Productivity - National Performance Indicator**

**KWINANA_FWY NORTH, MITCHELL_FWY SOUTH - AM Peak 6am - 10am**

**Jan - Mar 2008 to Jan - Mar 2009**

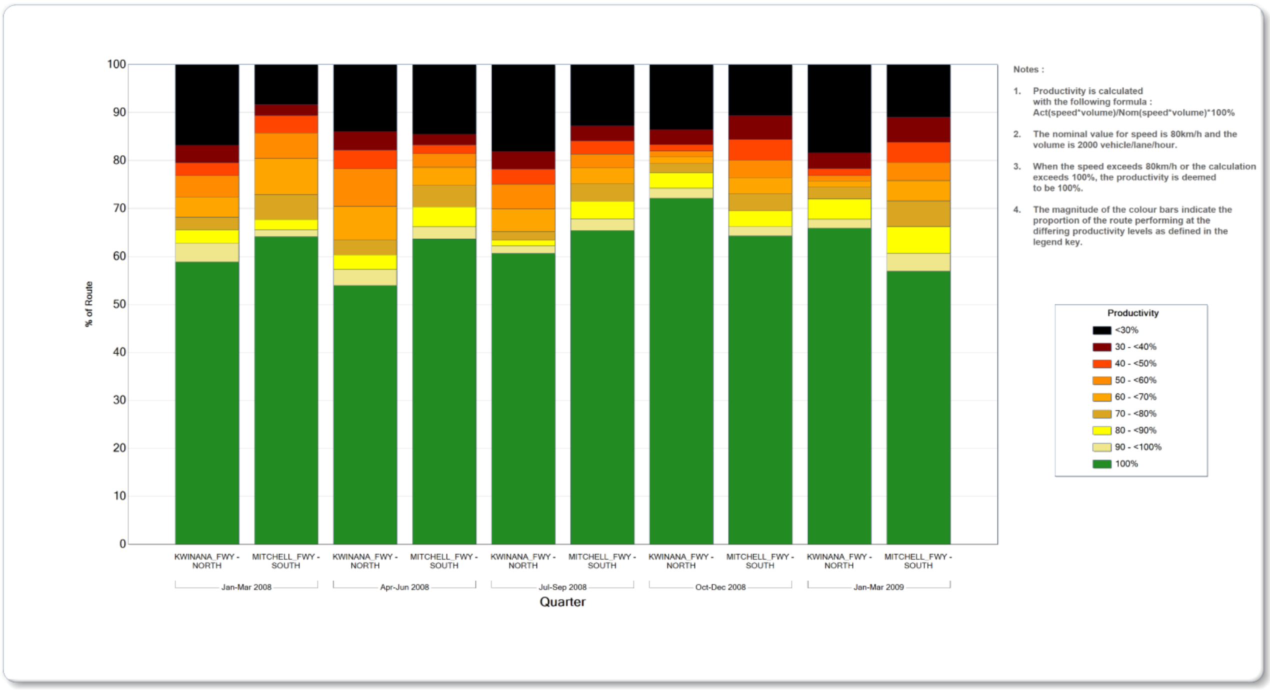



Figure 5: National Performance Indicator report comparing the Productivity of Kwinana Freeway (north bound) and Mitchell Freeway (south bound) during the morning peak period between 6am and 10am.

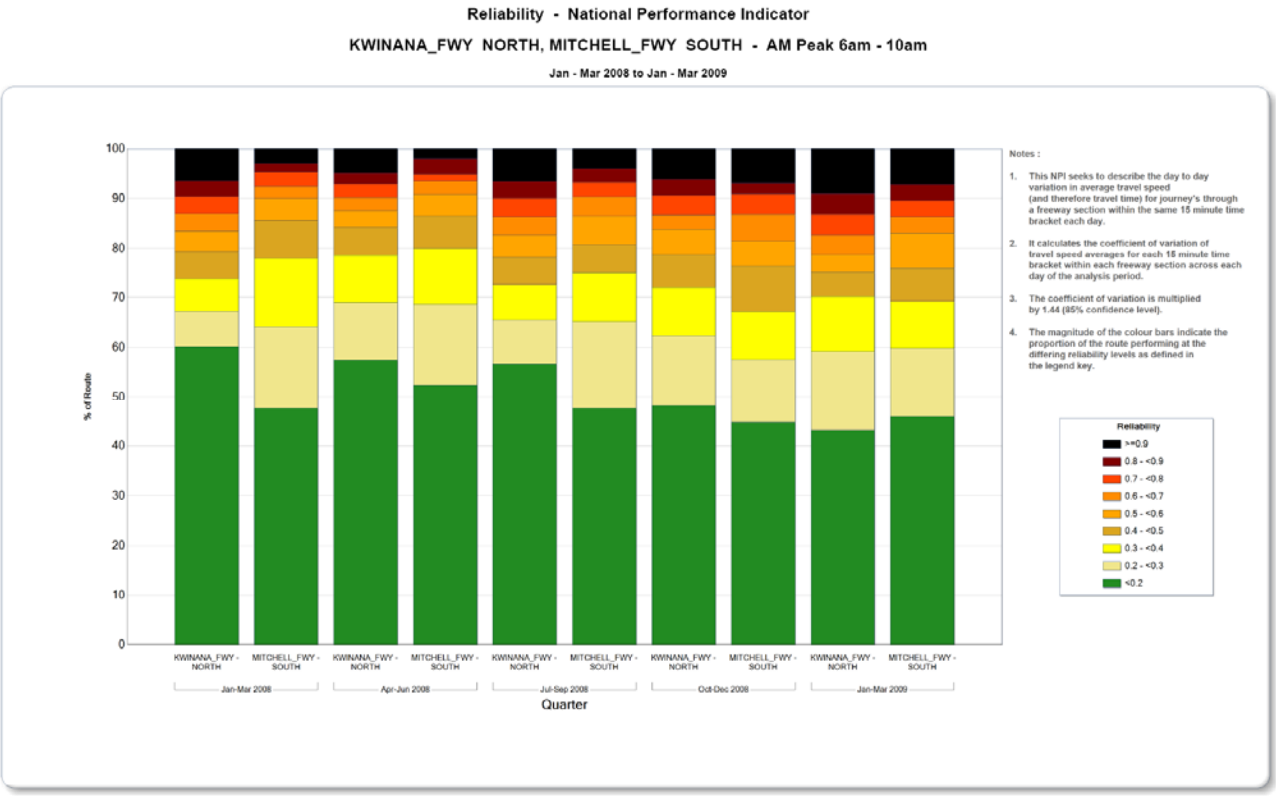



Figure 6: National Performance Indicator report comparing the Reliability of Kwinana Freeway (north bound) and Mitchell Freeway (south bound) during the morning peak period between 6am and 10am.

Main Roads is developing similar reports for the “direct” costs of travel and congestion associated with estimated travel time costs, reliability time costs, fuel consumption and vehicle emissions (e.g. greenhouse gas emissions). In addition, Main Roads is developing other methods for presenting the information, such as Graphical Information System (GIS) presentation of the road network, with colouring of the road network corresponding with the appropriate performance band.

## 3. ARTERIAL NETWORK TRAFFIC SIGNAL DATA

Main Roads presently uses the Sydney Coordinated Adaptive Traffic Signal (SCATS) system to manage the real time operations associated with all traffic signals in Western Australia.

While the loop detectors and Traffic Signal Controllers register every detector activation and deactivation, only summarised information is used by SCATS for signal phase optimisation and coordination. The summarised data is based upon “Strategic Approaches” which represent traffic lane groupings that provide the most important data for the signal phase optimisation and coordination. Therefore, the dataset available from the Strategic Approaches is not comprehensive and highly summarised, reducing the

quality of analysis that can be achieved. Despite the reduced outcome quality, the dataset that is available can be used for advanced analysis purposes such as the estimation of travel times and determination of traffic densities for the relevant traffic lanes[3].

Main Roads is seeking to develop the capability to extract this information from the SCATS system and apply an automated process that provides advanced analysis outputs for real time and historical network performance reporting. Investigation is proceeding to determine appropriate processes and assumptions in order to generate a relatively useful and accurate picture of network performance.

Upon system implementation, the level of accuracy is intended to be confirmed by comparing floating vehicle datasets with the system outputs. If a suitable level of accuracy is found, then the output will still form a basis for estimation of Sustainability Indicators associated with travel time and environmental emissions. Accordingly, it is a project objective to provide the capability for the reporting of indicators that are similar to the freeway, although weaker in accuracy and detail.

## 4. FLOATING VEHICLE DATA

Floating Vehicle Data (FVD) describes network performance data retrieved from vehicles travelling the road network. It may be acquired through fleet management systems, usually involving the Global Positioning System (GPS), mobile phone tracking systems, electronic tag tracking systems (e.g. toll operators) or vehicle recognition systems (e.g. Automatic Licence Plate Recognition and electro-magnetic vehicle signature measurement).

FVD has the potential to provide information about what is happening between traditional infrastructure based vehicle data systems (e.g. inductive loops), through locations without traditional vehicle data infrastructure and (in the case of the GPS fleet management systems) potentially with a high data frequency; however, there are issues associated with data ownership, reliability and actual data frequency. Firstly, while some data owners are pleased to pass their data to the road authority, initial investigation has found that many are not. Secondly, by normal measure, FVD has very high reliability, however, many of the systems utilise mobile phone communication pathways or systems, which can be prone to failure during important occasions such as major incidents and events, which is arguably when performance data is needed most. Finally, to reduce

[3] P. Bennett, J. Luk, B. Marsh, Real Time Estimation of Travel Time on Arterial Roads – The ARRB Travel Time Model, CAITR, December 2008.

costs, maximise battery life (e.g. mobile phone sampling) or to operate within available bandwidths, many FVD sets have relatively low sampling frequencies, reducing the ability to rapidly characterise network performance issues. While future advances are likely to address many of these issues, it is proposed that data source diversification provides the best solution.

MRWA presently has limited access to FVD. In the case of fleet management, few vehicles are equipped with this capability. In the case of mobile phone data, there remain questions in association with the data reliability plus there is a cost involved in acquiring the data. In the case of other vehicle tracking systems, infrastructure for this purpose (e.g. toll roads) does not presently exist. By contrast, MRWA is close to having significant VDS coverage of the freeway network and access to all traffic signal data. Accordingly, the current focus is upon using available FVD data, manual FVD survey's, to assist with the calibration of algorithms that estimate network performance based upon the VDS and Traffic Signal data sets.

A significant advantage provided by the MRWA approach is that the VDS and Traffic Signal data sets are already reliable and are becoming more reliable. They are already reliable because their communication pathway is not significantly prone to failure as it is commonly based upon an isolated network. Further reliability is emerging through the application of Unlimited Power Supply (UPS) systems at key locations (particularly Traffic Signals) and through the development of diverse communication pathways. For example optic fibre communication rings are presently being deployed through Foundation ITS projects and programs – see Figure 7.

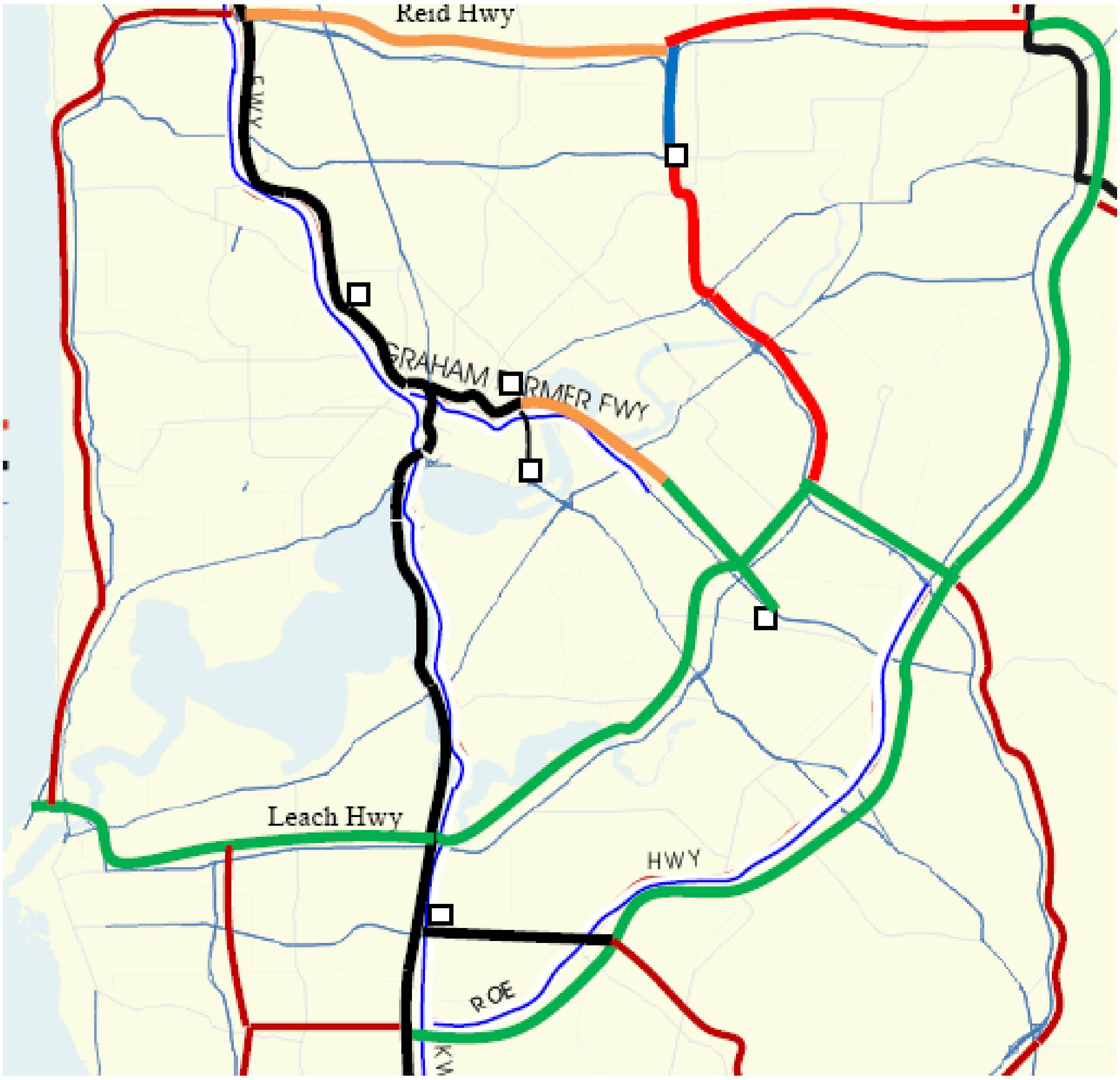



Figure 7: An excerpt from MRWA optic fibre planning process highlighting multiple optic fibre rings are being formed to provide diverse communication pathways, increasing the reliability of the communications system.

Supporting this approach is assessment of three key arterial road network roads of Reid Highway, Tonkin Highway and Leach Highway for accuracy of travel time estimation based upon the available Traffic Signal data. The algorithms were manually calibrated using FVD collected for this purpose. It was found that high levels of travel time estimation accuracy could be achieved through this process at route level with the correlation co-efficient and coefficient of determination exceeding 0.9[4]. The only deficiency associated with the trial exercise was the need to recalibrate the system over time to cope with amendments to Traffic Signal infrastructure, road infrastructure and operational regimes. To overcome this limitation and a process is being developed for

[4] P. Bennett, J. Luk, B. Marsh, Real Time Estimation of Travel Time on Arterial Roads – The ARRB Travel Time Model, CAITR, December 2008.

the automatic update of the system calibration, forming part of the automated system for extracting travel time information for the arterial road network alluded to in Section 3.

## 5. NETWORK VIEW OF PERFORMANCE

Section 3 notes the intention for arterial road data extraction based upon the Traffic Signals and SCATS system to be analysed to produce outputs similar to the freeway data. Section 4 refers to the application of available FVD to enable the automated calibration of systems used to estimate travel time, particularly based upon the Traffic Signal data. It is further noted that a similar process is proposed for the freeway data sets.

However, there are roads which exhibit characteristics of both freeway and arterial roads as they include grade separation at some intersections, sometimes long distances between intersections (e.g. proposed freeways) and retain traffic signals at some (if not all) intersections. Further, the true freeway network interacts with the arterial road network at interchanges, which involves a transition between freeway and Traffic Signal data sets.

To enable a single picture of network performance to be efficiently formed, it is essential that the alternative data sets be combined. Accordingly, MRWA is developing such a process.

Initially the process will involve generalisations and assumptions that are not always accurate at the micro level, however, provide a relatively accurate picture at the macro level. An example of a generalisation is associated with alternative pathways from a single traffic lane at a set of traffic signals; where arrows are painted on the road to define the pathway options (according to the Traffic Signal records in SCATS) traffic volumes will be initially allocated equally across those pathways. Clearly, this generalisation is at great risk of being inaccurate when a particular traffic signal set is considered at a selected time of the day. However, the fact that the arrows have been identified, compared to a circumstance where they have not, represents recognition that the pathway options are all important and so are each likely to receive significant traffic flows. It is a matter of probability as to which is the more significant flow at the localised scenario rather than a consistent trend. Accordingly, at the macro level, the generalisation is considered to provide a relatively accurate representation.

The system will enable the generalisations and assumptions to be adjusted for individual circumstances, which is important given the system will generate priorities for

improvement investigation (e.g. to consider operational improvement of the Traffic Signal coordination). Should the investigation reveal that the deficient performance at a location does not represent reality due to an assumption or generalisation being too inaccurate, then adjustment can be made to the specific location.

The final feature of the system is that it will involve a data cube structure for the historical analysis. This feature will enable prompt assessment of priorities and rapid reassessment as many system adjustments occur. While the cube will also have constraints, such as rules associated with the dimension settings are more difficult to adjust, the cube will enable faster processing and report building due to the advanced data organisation.

## 6. CONCLUSION

The Network Intelligence Project of MRWA is an exciting project that is developing capability in advanced data sets for Transportation and Traffic Engineering. Key features include sustainability performance indicators, automated reporting systems, detailed technical analysis, integration of diverse data sets, application of data cube technology and real time performance information publishing.

While the Network Intelligence Project is far from complete, significant results have already been achieved, particularly in the freeway Vehicle Detection Station area and the historical data analysis context. These results are positioning MRWA well in relation to achieving the benefits of ITS.

Further development stages include traffic signal data (through the SCATS), application of Floating Vehicle Data (to calibrate algorithms estimating network performance between VDS and traffic signals) and development of a unified “cube” that provides a network view of performance.

# The Application of Routing Planning on the Fieldwork Investigation

**Sz-Chi Chen, Tung-Ling Wu, Yu-Fen Ho**
*Institute of Transportation, Ministry of Transportation and Communications*
*Taipei 105, Taiwan, R.O.C.*
szchi@iot.gov.tw

**ABSTARCT**

Geographic Information System (GIS) integrates different types of digital geographic information. To maximize the information sharing and reduce the unnecessary duplication, the government departments should collect data and set up the data standard, quality requirement and updating system. Take "Institute of Transportation Traffic Road Network Digital Atlas" for example, the establishment and updating references include topographic base maps, aerial photography, satellite images and GPS vehicle tracking maps etc. Beside the spatial data as described above, the attribute data (ex. one way, turning restriction, speed limit, etc) usually is acquired through different management agencies or the fieldwork investigation which is most time and labor consuming.

The fieldwork investigation has been widely applied now, but most applications are not considering routing planning systematically. The efficiencies of the fieldwork investigation could be enhanced if we could find a minimum cost route for collecting data, subject to the specific nodes or links in network. This problem can be regarded as an extension of the Traveling Salesman Problem (TSP) and Chinese Postman Problem (CPP). In this study we propose modeling and heuristics solution for the fieldwork investigation. Numerical problems and real applications in "Institute of Transportation Traffic Road Network Digital Atlas" are provided for demonstration as well.

## 1. INTRODUCITON

The Road Network Digital Atlas data can be directly accessed via government departments or be generated by aerial photography; however, as to data-collecting, there are still situations that we cannot fully control such as low returning rate and low complement of research data, or problems flowing out when being checked. In order to overcome these situations, the fieldwork investigation is applied in this study for higher standard of research validity. The fieldwork investigation can be classified into two categories: (1) Differential Global Positioning System (DGPS) can collect the spatial data fast but the attribution data hardly. (2) The fieldwork investigation collects the attribute data (ex. one way, turning restriction, speed

limit, traffic lane, road shoulder, road name, etc).

The fieldwork investigation has been widely applied now, but most applications are not considering routing planning systematically. The efficiencies of the fieldwork investigation could be enhanced if we could find a minimum cost routing for collecting data, subject to the specific nodes or links in network. This problem can be regarded as an extension of the Traveling Salesman Problem (TSP) and Chinese Postman Problem (CPP). In this study we propose model to the above problem via set up an artificial arc and node to convert ARP into NRP. In order to simplify the complexity, we assume that:

1. Network:

(1) Cluster problem is not considered in this research; hence, we assume that every district would be has professional investigation troop.

(2) The specific nodes or links in network are known.

2. Cost:

(1) Cost estimation is based on the total travel time spent on investigation.

3. Investigation troop:

(1) The number of vehicles is given.

This study is divided into several sections. After this introductory section and the literature review in Section 2, a model is formulated and discussed in Section 3. In Section 4 we propose a heuristic algorithm. In Section 5, we demonstrate our model based on test problems. At the end of Section 5, we test our model on an actual case in Taipei—“Institute of Transportation Traffic Road Network Digital Atlas”.

## 2. LITERETURE REVIEW

The literature reviews on Road Network Digital Atlas and the fieldwork investigation are examined, followed by the reviews on routing problems. However, there are little researches on mixed problems of TSP and CPP are too scarce to be discussed. Therefore, we put more emphasis on the conversion from Arc Routing Problems (ARP) to Node Routing Problems (NRP).

### 2.1 Road Network Digital Atlas and the fieldwork investigation

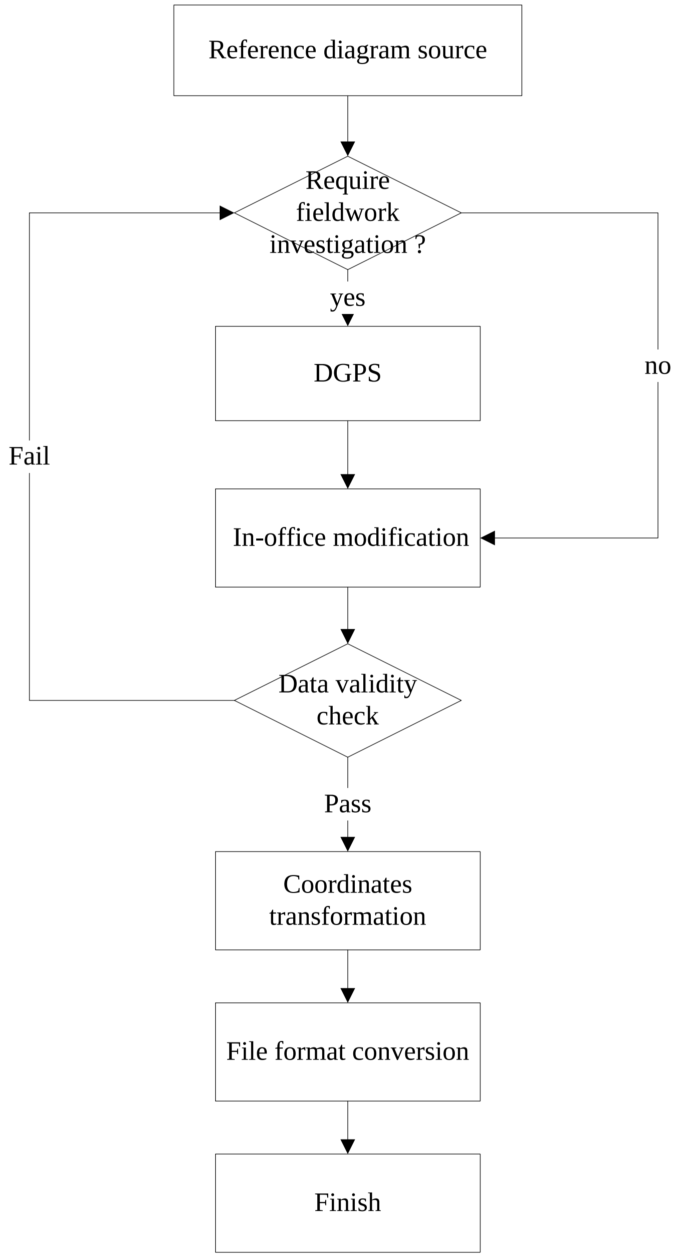



Figure 1. Road Network Digital Atlas creation/updating flow chart [1]

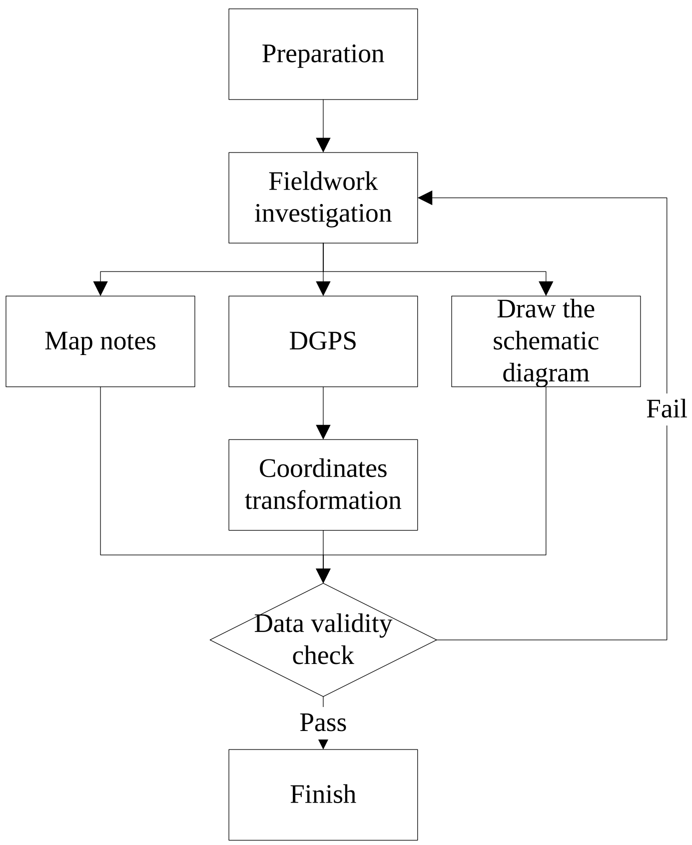



Figure 2. The fieldwork investigation flow chart [1]

Figure 1 indicates the flow chart of generating Road Network Digital Atlas. In this diagram, reference resources are checked the fieldwork investigation is required. Through out the whole process, data validity check would be preceded till it pass and later the validated data would be converted into the right data format and filed. Figure 2 shows more details of the fieldwork investigation process. However, route planning had not been took in these studies.

Arc routing related information can be obtained with ease via aerial photography and DGPS. Nevertheless, a number of streets and roads are still remained unknown due to the difficulties of obtaining the right attribute data. In order to solve this problem, the fieldwork investigation application is required.

## 2.2Routing problems

Practically many routing problems require a certain route to visit the specific nodes in certain network, which can be broadly defined as Node Routing Problem (NRP) and Arc Routing Problem (ARP). These problems are pretty common in daily lives. The most well-known routing problems are Traveling Salesman Problem (TSP) and Chinese Postman Problem (CPP).

The TSP is a problem in combinatorial optimization studied in operations research

and theoretical computer science. Given a list of cities and their pairwise distances, the task is to find a shortest possible tour that visits each city exactly once.

The CPP is a problem to determine a minimum length walk covering each segment at least once. The CPP can be categorized into three types based on the direction of arcs in the network: (1) Undirected Chinese Postman Problem (UCPP). (2) Directed Chinese Postman Problem (DCPP). (3) Mixed Chinese Postman Problem (MCPP).

A generic ARP that subsumes a number if well-known routing problem is the Capacitated Arc Routing Problem (CARP). This problem, introduced by Golden and Wang, may be briefly defined as follows: Given a network where costs and demands are associated with each arc, find a set of minimum cost vehicle cycles, based at a distinguished node call the depot, and traversing all arcs of positive demand, so that the total demand serviced by each cycle does not exceed the vehicle capacity $W$ .As pointed out by Golden and Wong, a number of well-known node routing problems can be viewed as special cases of the CARP simply by splitting any original node into two nodes joined by an arc, and assigning the demand of the original node to the arc.

The CARP can be formulated as a standard vehicle (node) routing problem by Pearn et al.(1987)[5]. They transform ARP into NRP by three new nodes, shown as Figure. 3. and the directivity is $s_{ij} => m_{ij} => s_{ji}$ or $s_{ji} => m_{ij} => s_{ij}$ .Aminu and Eglese(2006)[8] was inspired by Pearn et al.(1987) and modified the concept. They transform ARP into NRP by two new nodes, shown as Figure. 4., the directivity and time windows are considerate. In their study, Total cost and the sequence of route can be obtained but part of the distance matrix between the new nodes (calculated with shortest path) is inaccuracy ( Sz-Chi Chen, 2007)[3].

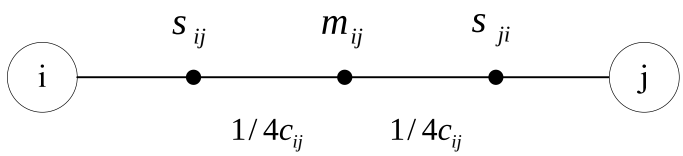



Figure 3. Transforming CPP to NRP [5]

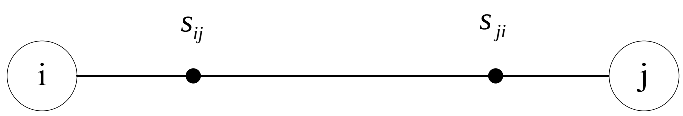



Figure 4. Modification of Pearn et al.'s approach [8]

DCPP is P (polynomial) problem (Edmonds and Johnson, 1973[9]) and can be obtained the solution quickly by minimum cost flow, Lin and Zhao algorithm. But there are situations of multiple solutions and we can make sure the sequence of the route.

## 3. MODEL FORMULATION

The fieldwork investigation can be regarded as an extension of TSP and CPP. Literature reviews on these kinds of problems were often proposed as artificial nodes, which converts ARP into NRP. In this study, we try to set up artificial arcs and nodes to convert the problem visiting certain nodes and arcs of a network, and we hope through this research, we can conclude that the mixed problems are equal to TSP.

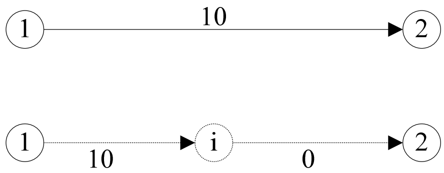



Figure 5. Transforming CPP to TSP

As shown in Figure 5, the artificial arc and node can replace the original arc, and the cost between the node 1 and the node $i$ is the original arc cost 10; the cost between the node $i$ and the node 2 is cost 0. Hence, the mixed problem can be regarded as an extension of the TSP that we can and can be formulated as a mixed integer problem, as follows:

$$\min_{\{x_{ijk}\}\in\Omega} Z(\tau) = \sum_i \sum_j \sum_k \{c_{ij}x_{ijk} + c_{0j}x_{0jk}\} \tag{1}$$

Flow conservation constraints:

$$\sum_j \sum_k x_{ijk} = 1 \qquad \forall i \in N, k \in K \tag{2}$$

$$\sum_i \sum_k x_{ijk} = 1 \qquad \forall j \in N, k \in K \tag{3}$$

$$\sum_i x_{ihk} - \sum_j x_{hjk} = 0 \qquad \forall h \in N, k \in K \tag{4}$$

$$x_{0ik}, x_{ijk} \in \{0,1\} \qquad \forall i, j \in N, k \in K \tag{5}$$

Definitional constraints:

$$a_j = d_i + c_{ij} \qquad if \quad x_{ijk} = 1 \qquad \forall i, j \in N, k \in K \tag{6}$$

$$a_j = c_{0j} \qquad if \quad x_{0jk} = 1 \qquad \forall j \in N, k \in K \tag{7}$$

$$\sum_i \sum_j \{c_{ij}x_{ijk} + c_{0j}x_{0jk} + s_i x_{ijk}\} \le work\ time \quad \forall k \in K \tag{8}$$

The objective of this model, as shown in Eq. (1), Eq. (2)~(4) are flow conservation constraints: Eq. (2) requires that only one vehicle $k$ can leave from node $i$ once. $k$ is belong to Investigation troop. Eq. (3) denotes that only one vehicle can arrive at node $j$ once. Eq. (4) states that for each node h, the entering vehicle must eventually leave this node. Eq. (5)

designates $x_{ijk}$ as a 0–1 integer variable. $x_{ijk}$ equals 1 if vehicle $k$ departs node $i$ toward node $j$. Otherwise, $x_{ijk}$ equals 0. Eq. (6) and (7) define the arrival time at node j. Eq. (8) denotes that every vehicle has working-time limit.

The departure time at each node is treated as a decision variable with which there is no need to impose numerous sub-tour constraints. In ordinary cases, there are situations of multiple solutions, as shown in Figure 6. Nodes and arcs to be investigated are shown as dotted and replaced by the artificial ones. The distance between artificial nodes should be the shortest path cost. Take Figure 6 for instance. There is more than one solution: (1)0→1→2→3→4→5→0. (2) 0→1→3→2→4→5→0. (3) 0→1→4→5→3→2→0. (4) Etc. With no other constraints or time windows, there are often multiple solutions to the problems. This study sets the investigation time $s_i$ as a constant and the arrival time as a variable. Thus, the algorithm can come up with one answer, such as (1) 0→1→2→3→4→5→0, and in practice artificial node 2 would only be calculated once at the first time, which means the investigation would only be held once- first pass and pass by at the second time .

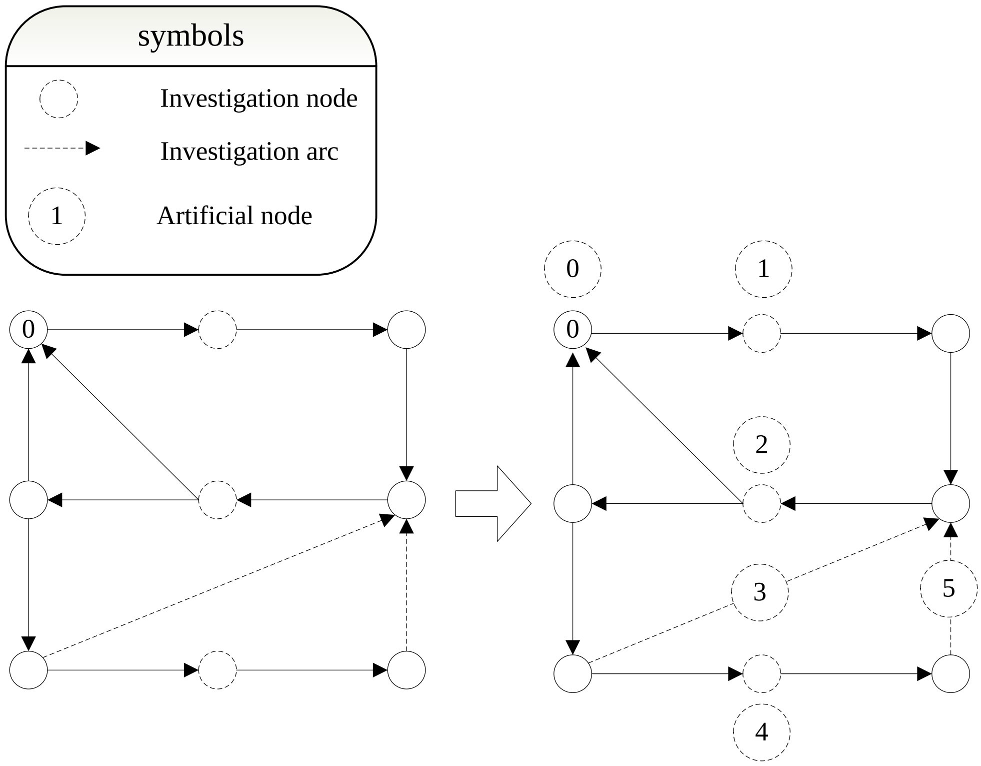



Figure 6. Network example

## 4. SOLUTION ALGORITHM

We divide the algorithm into two phases. Phase I is deal with mixed problems of TSP and CPP. In phase I We transform CPP to TSP and Compute shortest path cost of every artificial nodes and generate input data. Phase II is to solve TSP. In phase II the insertion method is used for route construction, while the Or-opt node exchange algorithm is adopted for route improvement. Then the algorithm can come up with one answer of Phase II, and we convert

the result into an original route.

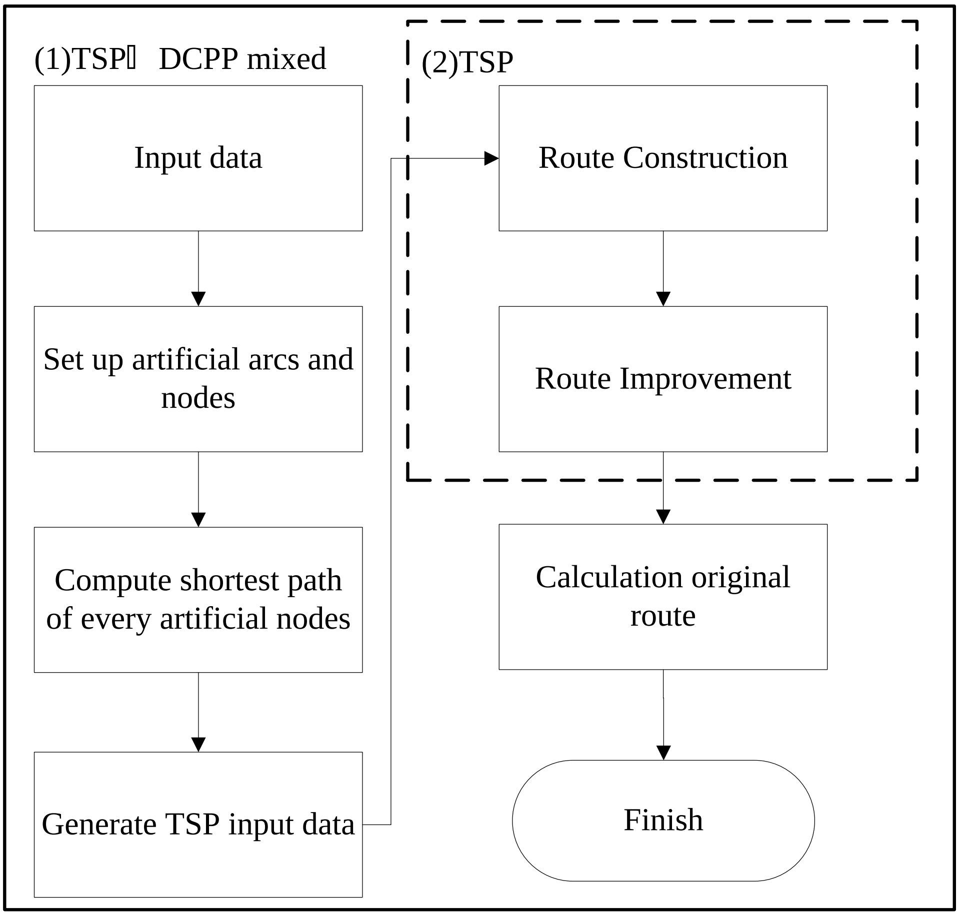



Figure 7. Unified framework of solution procedure

## 5. NUMERICAL EXAMPLES

### 5.1 Examination of Numerical examples

In order to demonstrate the conversion from ARP to NRP in this study, we compare results that are base on Lin and Zhao algorithm, our algorithm, and LINGO9.0. The criterions are the total travel time and number of arcs in the five examples. To begin with Case (1), the result is shown below as Figure 8: (Node 1 is depot).

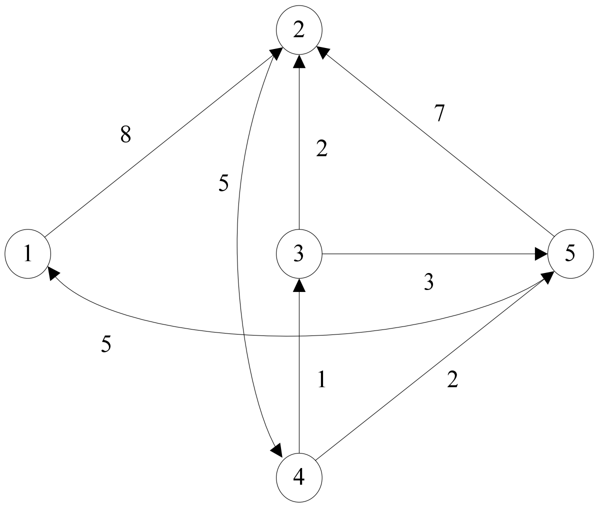



Figure 8. Case(1) network

a. Lin and Zhao

Code is based on Lin and Zhao’s algorithm.

Routes:

(1) 1→2→4→3→5→2→4→3→2→4→5→1.

(2) 1→2→4→5→2→4→3→2→4→3→5→1.

(3) 1→2→4→3→2→4→3→5→2→4→5→1.

**Table 1**.Case (1) Lin and Zhao result

| Arc | Cost r(i,j) | Number of Arcs h(i,j) | w(i,j) |
|---|---|---|---|
| 1,2 | 14 | 1 | 14 |
| 2,4 | 0 | 3 | 0 |
| 3,2 | 8 | 1 | 8 |
| 3,5 | 4 | 1 | 4 |
| 4,3 | 0 | 2 | 0 |
| 4,5 | 2 | 1 | 2 |
| 5,1 | 4 | 1 | 4 |
| 5,2 | 12 | 1 | 12 |
| Total cost | 44 | | |
| Total number of arcs | | 11 | |

b. Our algorithm

The algorithm of has fully demonstrated in the previous section. First, we transform ARP into NRP. Then we set no investigation time ($s_i = 0$) for comparison with Lin and Zhao algorithm, LINGO9.0. The result is shown below as Table 2 and we convert the result into an original route.

**Table 2.**Case (1) algorithm input

| artificial node | forward node | toward node | investigation time $s_i$ |
|---|---|---|---|
| 1 | 1 | 2 | 0 |
| 2 | 2 | 4 | 0 |
| 3 | 3 | 2 | 0 |
| 4 | 3 | 5 | 0 |
| 5 | 4 | 3 | 0 |
| 6 | 4 | 5 | 0 |
| 7 | 5 | 1 | 0 |
| 8 | 5 | 2 | 0 |

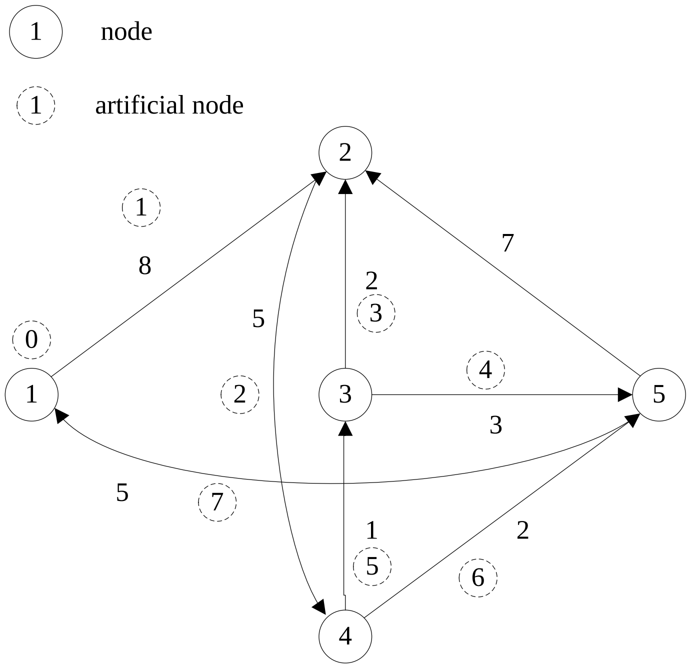



Figure 9. Case(1) artificial network

**Table 3.**Case (1) algorithm result

| artificial node | arrival time $a_i$ | departure time $d_i$ | cost $c_{ij}$ | investigation time $s_i$ |
|---|---|---|---|---|
| 0 | 0 | 0 | 8 | 0 |
| 1 | 8 | 8 | 9 | 0 |
| 4 | 17 | 17 | 7 | 0 |
| 8 | 24 | 24 | 6 | 0 |
| 5 | 30 | 30 | 2 | 0 |
| 3 | 32 | 32 | 5 | 0 |
| 2 | 37 | 37 | 2 | 0 |
| 6 | 39 | 39 | 5 | 0 |
| 7 | 44 | 44 | 0 | 0 |
| 0 | 44 | 44 | 0 | 0 |

Total time：44 min.

Total number of arcs：11.

Artificial node route：0→1→4→8→5→3→2→6→7→0(0 denotes depot.)

Original route：1→2→4→3→5→2→4→3→2→4→5→1(1 denotes depot.)

LINGO9.0

Total time：44 min.

Total number of arcs：11.

(Global Optimum)

**Table 4**.Case (1) LINGO9.0 result

| Variable | Value | Reduced Cost |
|---|---|---|
| X( 1, 2) | 1 | 8 |
| X( 1, 2) | 3 | 5 |
| X( 1, 2) | 1 | 2 |
| X( 1, 2) | 1 | 3 |
| X( 1, 2) | 2 | 1 |
| X( 1, 2) | 1 | 2 |
| X( 1, 2) | 1 | 5 |
| X( 1, 2) | 1 | 7 |

**Table 5**.Result comparisons

| Case | Lin and Zhao | | Our algorithm | | LINGO9.0 | |
|---|---|---|---|---|---|---|
| | Objective value | Total number of arcs | Objective value | Total number of arcs | Objective value | Total number of arcs |
| (1) | 44 | 11 | 44 | 11 | 44 | 11 |
| (2) | 73 | 14 | 73 | 14 | 73 | 14 |
| (3) | 45 | 12 | 45 | 12 | 45 | 12 |
| (4) | 88 | 15 | 88 | 15 | 88 | 15 |
| (5) | 29 | 8 | 29 | 8 | 29 | 8 |

In view of several different numerical examples, algorithm can be applied in the small scale example very well. After transforming the result into original route, we can get total cost, the number of arcs, and arrival time of each artificial node that we can know the sequence.

5.2 Examination on Road Network Digital Atlas

For testing modeling and algorithm in large-scale network, Taipei city network, from Road Network Digital Atlas, produced by Institute of Transportation (IOT) is chosen to be the basis of this examination, shown as Figure 10 and 11, which aims to investigate the major road flows in Taipei. The assumptions are as followed:

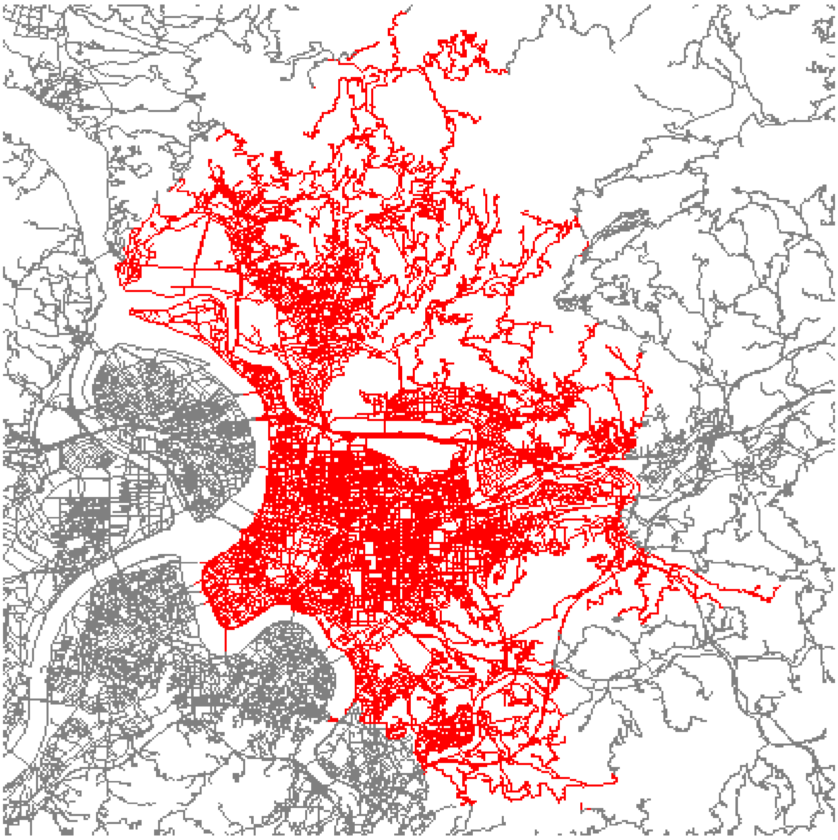


Figure 10. Network of Taipei city

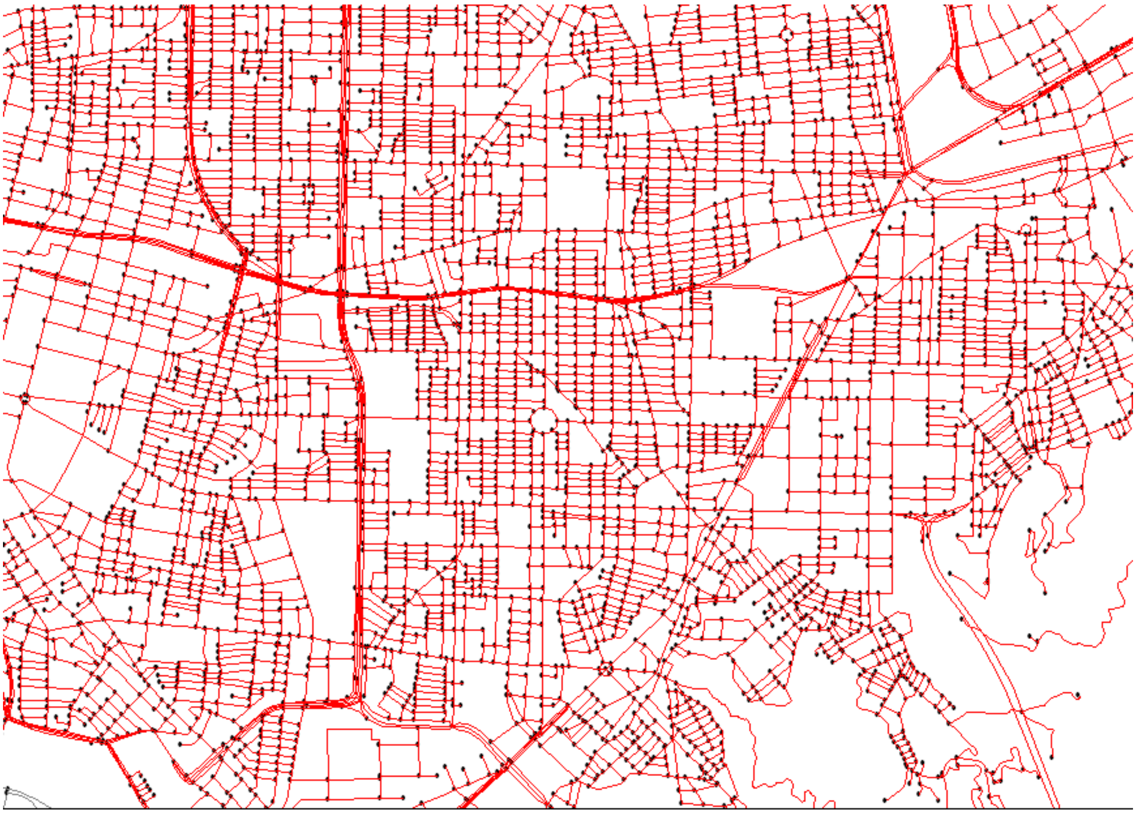


Figure 11. Street network of Taipei city

(1) Single depot, which generates 275 nodes and arcs to be investigated; the investigation time is assumed to be 1 minute.

(2) The investigation work troops are 10 vehicles at the maximum and each of them is under work hour constraints. However, in reality every vehicle takes quite a plenty of time to complete the whole investigation, and practically speaking, usually one vehicle is distributed to every working troop. Therefore, we might take the number of working days into consideration. Based on the working-time limit given, 10 vehicles should be able to work for 10 days as defined.

(3) Working-time limit are set as following: 2 hours, 4 hours, 6 hours, 8 hours and unlimited.

From Table 6 and Figure 12, we can know that working-time limit should be set when having large-scale network investigation; otherwise unreasonable working time, such as case (6), might occur. Meanwhile, total travel time does not change a lot under different work-hour limit; on the other hand, the number of vehicles used changes dramatically. If the fieldwork investigation range is larger, more vehicles are required. We could use several vehicles or use many working days with a vehicle to complete the fieldwork investigation. Case 2, 3 (work time limit: 3hours, 4hours) are more reasonable than others. And the route result of case 3 is shown

in Appendix.

**Table 6.**Cases Comparison

| Case | (1) | (2) | (3) | (4) | (5) | (6) |
|---|---|---|---|---|---|---|
| Working-time limit(min) | 120 | 180 | 240 | 360 | 480 | unlimited |
| Total number of vehicles | 6 | 4 | 3 | 2 | 2 | 1 |
| Total(min) | 427.66 | 426.81 | 426.32 | 418.01 | 418.58 | 405.48 |

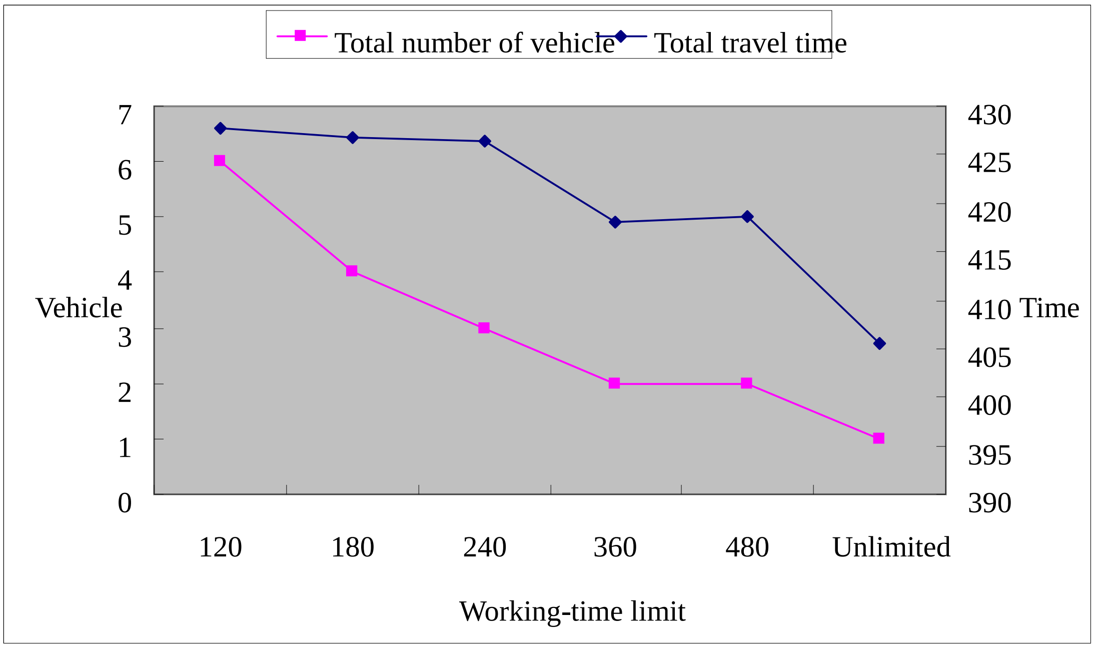



Figure12. Cases comparison

# 6. CONCLUSION

The value of Road Network Digital Atlas is sustained the maintenance and update of the data. Therefore, obtaining the up-to-date data is vital. Also, the fieldwork investigation not only checks for problems but also update and expand data via data collection. We can reduce the labor consumption by routing planning systematically.

The conclusion of this study can be summarized as follows:

1. In this study, we try to set up artificial arcs and nodes to convert the problem visiting certain nodes and arcs of a network, and the fieldwork investigation can be regarded as an extension of TSP that we formulated as a mixed integer problem.
2. The traditional of TSP and CPP studies are lack of discussion on working-time limit. For practical applications, work time limit is considered in this study.
3. For testing modeling and algorithm, result comparisons of Lin and Zhao, LINGO9.0 are demonstrated. We also tested in large-scale network, Taipei city network, from “Institute of

Transportation Traffic Road Network Digital Atlas”. We provided some routes for the fieldwork investigation.

## REFERENCES


1. Institute of Transportation, Ministry of Transportation and Communications, 2003, “New Century National Transportation Road Network Digital Atlas.”
2. Institute of Transportation, Ministry of Transportation and Communications, 2007, “Maintaining and Updating the Digital Road Atlas.”
3. Sz-Chi Chen, 2007, “The Patrol Car Routing Problem.”
4. Lin, Y., and Zhao Y., 1988, “A New Algorithm for the Directed Chinese Postman Problem,” *Computers and Operations Research*, 15(6), pp. 577-584.
5. Pearn, W-L, Assad, A, Golden, B. L. 1987, “Transforming Arc Routing into Node Routing Problems,” Computers and Operations Research Vol. 14(4), 285-288
6. Golden, B. and Wong, R. T., 1981, “Capacitated Arc Routing Problems,” Network, Vol. 11, pp. 305-315.
7. Assad, A., Pearn, W. and Golden, B., 1985, “The Capacitated Chinese Postman Problem: Lower Bounds and Solvable Case,” MS/S Working Paper 85-032, University of Maryland.
8. Aminu, U.F. and Eglese, R.W., 2006, “A Constraint Programming Approach to the Chinese Postman Problem with Time Windows,” *Computers and Operations Research*, 33, pp. 3423-3431.
9. Edmonds, J., and E. L. Johnson, 1973, “Matching, Euler Tours and the Chinese Postman,” Mathematical Programming, Vol. 5, pp. 88-124.
10. Chen, H.K., Hsueh, C.F., and Chang, M.S., 2006, “The Real-time Time-dependent Vehicle Routing Problem,” *Transportation Research Part E*, Vol. 42, pp. 383-408.
11. Hsueh, C.F., Chen H.K., and Chou, H.W., 2007, “Vehicle Routing for Relief Logistics in Natural Disasters,” Working Paper at National Central University, Jungli, Taiwan.
12. Dror, M. 2000, Arc Routing: Theory, Solutions and applications, Boston: Kluwer Academic Publishers.


## APPENDIX

| Vehicle | Route |
|---|---|
| 1 | 0(0.0,0.0)--1(0.2,1.2)--3(1.4,2.4)--5(2.8,3.8)--29(4.1,5.1)--7(5.4,6.4)--28(6.6,7.6)--61(8.1,9.1)--72(9.3,10.3)--74(10.6,11.6)--78(12.2,13.2)--84(14.1,17.1)--86(17.4,18.4)--88(18.9,19.9)--90(20.1,21.1)--91(21.5,22.5)--93(22.7,23.7)--95(24.1,25.1)--103(26.6,27.6)--107(28.5,29.5)--109(29.7,30.7)--112(31.5,32.5)--115(33.0,34.0)--105(34.6,41.0)--94(41.8,42.8)--110(46.1,47.1)--82(48.6,49.6)--85(50.0,51.0)--92(51.3,52.3)--83(53.0,54.0)--97(54.3,55.3)--99(55.7,56.7)--100(57.1,58.1)--102(58.7,59.7)--96(60.1,61.1)--98(61.8,62.8)--79(63.7,64.7)--55(64.8,65.8)--56(66.2,67.2)--37(67.5,68.5)--57(68.8,69.8)--65(71.5,72.5)--67(72.8,73.8)--70(74.2,75.2)--73(75.5,76.5)--114(82.4,83.4)--242(85.5,86.5)--245(87.5,88.5)--253(89.5,90.5)--243(93.8,94.8)--236(95.5,96.5)--234(98.0,99.0)--232(99.3,100.3)--211(102.4,103.4)--207(104.1,105.1)--208(105.4,106.4)--210(107.5,108.5)--214(109.2,110.2)--230(111.1,112.1)--231(113.1,114.1)--235(114.5,115.5)--237(116.1,117.1)--240(118.6,119.6)--241(120.0,121.0)--87(121.4,122.4)--89(122.7,123.7)--81(124.9,125.9)--54(126.3,127.3)--75(128.4,129.4)--76(129.8,130.8)--77(131.1,132.1)--58(132.5,133.5)--66(135.0,136.0)--68(136.3,137.3)--20(141.3,142.3)--69(143.1,144.1)--71(144.4,145.4)--59(147.1,148.1)--35(151.7,152.7)--62(154.4,155.4)--26(157.3,158.3)--8(158.5,159.5)--9(159.7,160.7)--14(161.3,162.3)--16(162.7,163.7)--18(163.8,164.8)--11(166.5,167.5)--12(167.8,168.8)--15(169.3,170.3)--13(172.3,173.3)--25(174.0,175.0)--23(176.4,177.4)--22(178.6,179.6)--33(182.4,183.4)--63(185.5,186.5)--64(186.8,187.8)--19(188.2,189.2)--21(189.5,190.5)--36(192.3,193.3)--4(194.5, |

| | |
|---|---|
| | 195.5)--6(195.7,196.7)--10(197.1,198.1)--24(198.4,199.4)--27(199.7,200.7)--30(200.9,201.9)--31(202.3,203.3)--32(203.6,204.6)--34(205.1,206.1)--2(206.5,207.5)--44(207.8,208.8)--217(209.4,210.4)--47(211.2,212.2)--52(212.4,213.4)--53(213.6,214.6)--43(220.5,221.5)--49(222.4,223.4)--50(223.7,224.7)--39(229.8,230.8)--41(231.1,232.1)--45(232.4,233.4)--46(233.7,234.7)--48(235.0,236.0)--40(236.3,237.3)--42(237.6,238.6)--38(238.9,239.9)--0(239.9,0.0)-- total number of node : 124 |
| 2 | 0(0.0,0.0)--244(12.8,16.2)--249(18.6,19.6)--251(20.2,21.2)--252(22.3,23.3)--247(26.6,27.6)--233(29.3,30.3)--206(31.8,40.6)--196(41.4,42.4)--183(43.7,44.7)--225(47.0,48.0)--222(48.8,49.8)--223(50.2,51.2)--224(51.3,52.3)--17(53.3,54.3)--137(54.6,55.6)--136(56.0,57.0)--146(60.2,61.2)--148(61.5,62.5)--153(63.6,64.6)--157(64.9,65.9)--160(66.2,67.2)--149(70.4,71.4)--151(71.7,72.7)--155(73.0,74.0)--170(74.5,75.5)--166(76.8,77.8)--171(78.1,79.1)--172(79.8,80.8)--173(81.1,82.1)--175(83.7,84.7)--203(88.0,89.0)--116(96.0,97.0)--117(97.7,98.7)--118(99.1,100.1)--121(101.4,102.4)--262(103.0,104.0)--259(105.3,106.3)--261(107.2,108.2)--119(108.6,109.6)--120(110.3,111.3)--122(111.6,112.6)--124(112.9,113.9)--131(117.0,118.0)--133(118.3,119.3)--154(125.5,126.5)--152(130.1,131.1)--156(131.7,132.7)--158(133.1,134.1)--161(134.5,135.5)--162(135.8,136.8)--159(141.4,142.4)--163(143.6,144.6)--169(147.0,148.0)--164(149.3,150.3)--165(151.1,152.1)--167(152.5,153.5)--168(153.8,154.8)--150(155.7,156.7)--180(157.4,158.4)--200(163.3,164.3)--101(169.9,170.9)--128(171.4,172.4)--125(173.0,174.0)--127(174.7,175.7)--129(176.3,177.3)--130(178.0,179.0)--132(179.1,180.1)--134(180.5,181.5)--135(182.0,183.0)--138(183.5,184.5)--139(184.8,185.8)--140(186.4,187.4)--141(187.7,188.7)--142(189.2,190.2)--143(190.6,191.6)--144(191.9,192.9)--145(193.2,194.2)--147(195.0,196.0)--181(196.7,197.7)--182(198.3,199.3)--184(199.7,200.7)--185(201.6,202.6)--194(202.9,203.9)--195(204.4,205.4)--198(206.5,207.5)--201(208.4,209.4)--212(214.0,215.0)--197(216.1,217.1)--199(217.8,218.8)--204(219.2,220.2)--209(221.1,222.1)--215(222.4,223.4)--213(225.9,226.9)--227(227.9,228.9)--226(230.0,231.0)--228(231.6,232.6)--229(233.8,234.8)--238(236.8,237.8)--0(239.3,0.0)-- total number of node：98 |
| 3 | 0(0.0,0.0)--187(13.1,14.1)--216(18.6,19.6)--51(21.4,22.4)--80(25.2,26.2)--254(32.4,33.4)--275(39.4,40.4)--188(55.1,56.1)--190(56.9,57.9)--193(59.0,60.0)--239(63.9,64.9)--60(66.6,67.6)--106(71.5,72.5)--108(72.9,73.9)--104(74.6,75.6)--111(77.0,78.0)--113(78.5,79.5)--265(82.8,83.8)--270(87.1,88.1)--271(90.0,91.0)--273(93.1,94.1)--274(95.3,96.3)--264(98.4,99.4)--260(99.9,100.9)--266(101.6,102.6)--258(106.2,107.2)--268(108.1,109.1)--269(110.0,111.0)--255(112.1,113.1)--257(113.8,114.8)--267(116.5,117.5)--272(118.4,119.4)--256(125.7,126.7)--263(129.7,130.7)--123(131.9,132.9)--126(133.7,134.7)--174(146.3,147.3)--177(148.0,149.0)--178(149.2,150.2)--179(150.4,151.4)--218(165.9,166.9)--186(167.8,168.8)--191(169.9,170.9)--248(187.4,188.4)--250(188.8,189.8)--246(190.8,191.8)--189(210.6,211.6)--192(212.2,213.2)--202(215.6,216.6)--176(227.5,228.5)--219(229.6,230.6)--220(231.3,232.3)--221(232.8,233.8)--205(236.2,237.2)--0(239.7,0.0)--total number of node：53 |
| Working-time limit | 240 min |
| Total number of vehicle | 3 |
| Total time | 426.32 min |

# THE EFFECTS OF STEPPED-UP MEASURES FOR SLOWING DOWN THE SPEED IN THE ETC LANE

**Akira MITSUISHI , Hiroshi OGAWA, Masatoshi YOKOTA**

*East Nippon Expressway Company Limited*

*3-3-2 Kasumigaseki, Chiyoda-ku, Tokyo 100-8979 Japan*

a.mitsuishi.aa@e-nexco.co.jp

## 1. ABSTRUCT

For the purpose of ensuring safety for customers passing through ETC lanes, the East Nippon Expressway Company Limited (NEXCO East) has implemented stepped-up measures for slowing down the speed in ETC lanes throughout the company. According to the measures, the timing at which the toll bars open are delayed. This paper presents the results of the trial conducted ahead of the company-wide implementation of the measures. As a result, the speed has reduced drastically as well as the contacting rates with ETC toll bars. Further, in order to prevent the overall traffic flow from slowing down caused by the measures, NEXCO East conducted a trial in which customers such as drivers were visually notified of the timing at which the toll bars opened, and the effects of the trial are also introduced in this paper.

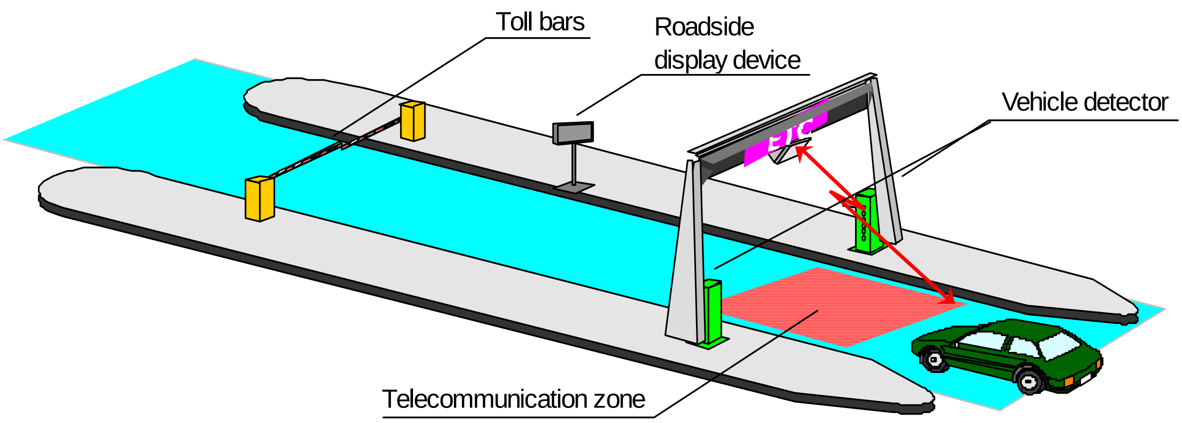



Figure 1. Overview of ETC lane

## 2. OVERVIEW OF THE ETC SYSTEM IN JAPAN

Since the full-swing introduction of ETC in Japan in 2001, ETC became widespread smoothly. As of the end of February 2009, some 77.1% of the vehicles on the nation's expressways use ETC. To collect expressway fees without fail with the use of the existing toll gates, toll bars are installed in the Japanese ETC lanes. However, since the toll bars remain closed without proper installation of the ETC card, a vehicle could collide with the toll bars or another vehicle that came to a stop in the ETC lane. Thus, vehicles need to slow down to 20 kilometres per hour or less in the ETC lane so that it can stop safely at any time. However, the speed in the ETC lane is relatively high, and the trouble caused in the lane still exists.

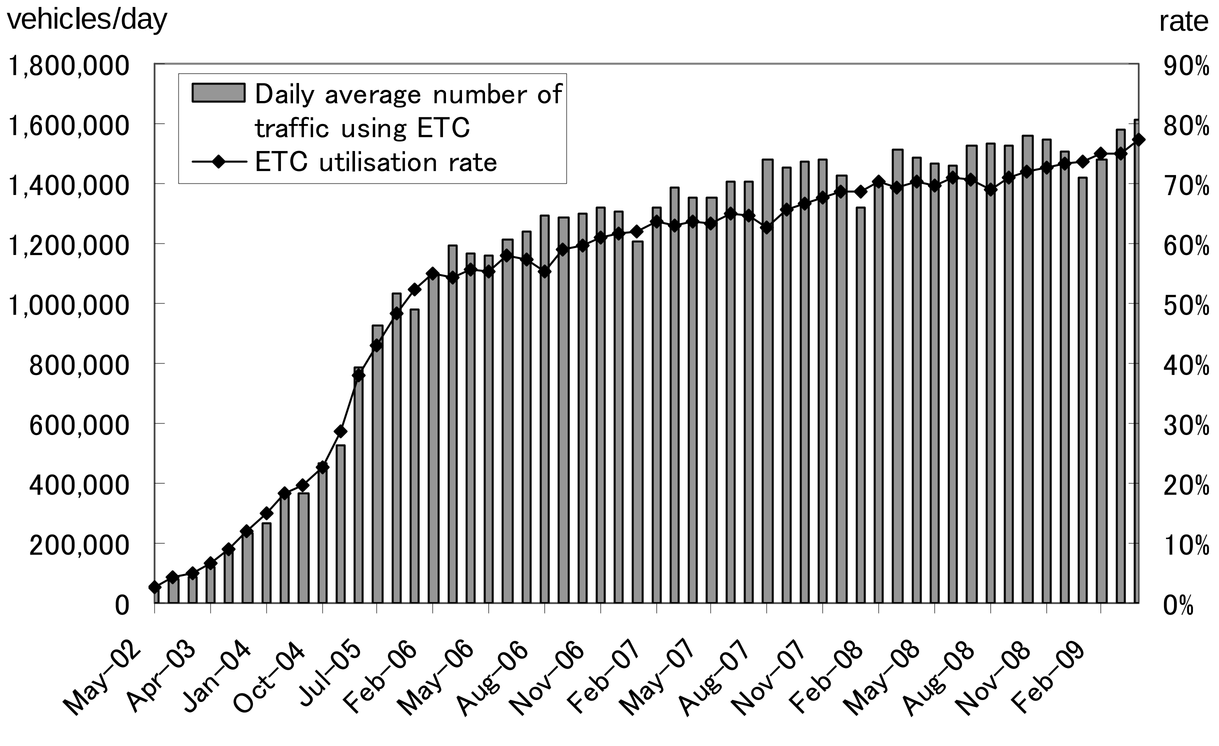



Figure 2. Transition of ETC utilisation rate

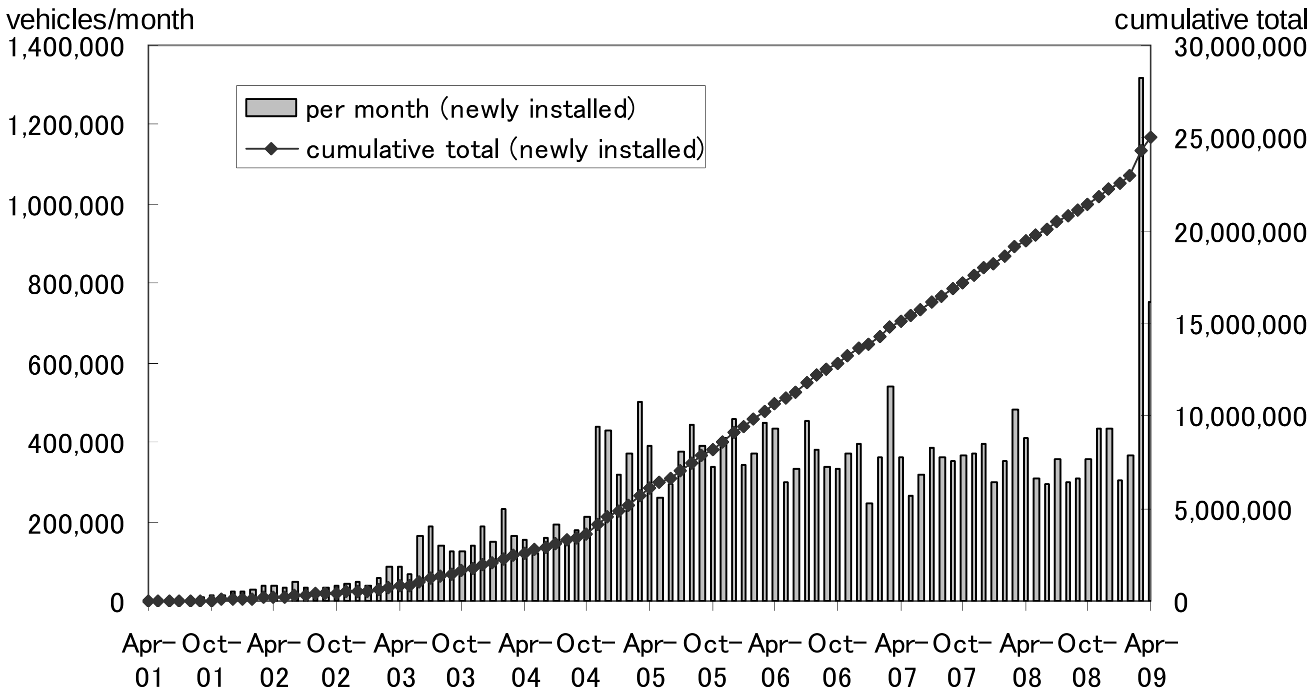



Figure 3. Transition of the number of vehicles with ETC devices

## 3. TROUBLE IN THE ETC LANE

### 3.1. Collision in the ETC lane (with objects/vehicles)

In the NEXCO East area, no fatal accidents have so far occurred in the ETC lanes. However, there seems to be no end to accidents that involve collision with toll bars or stopped traffic.

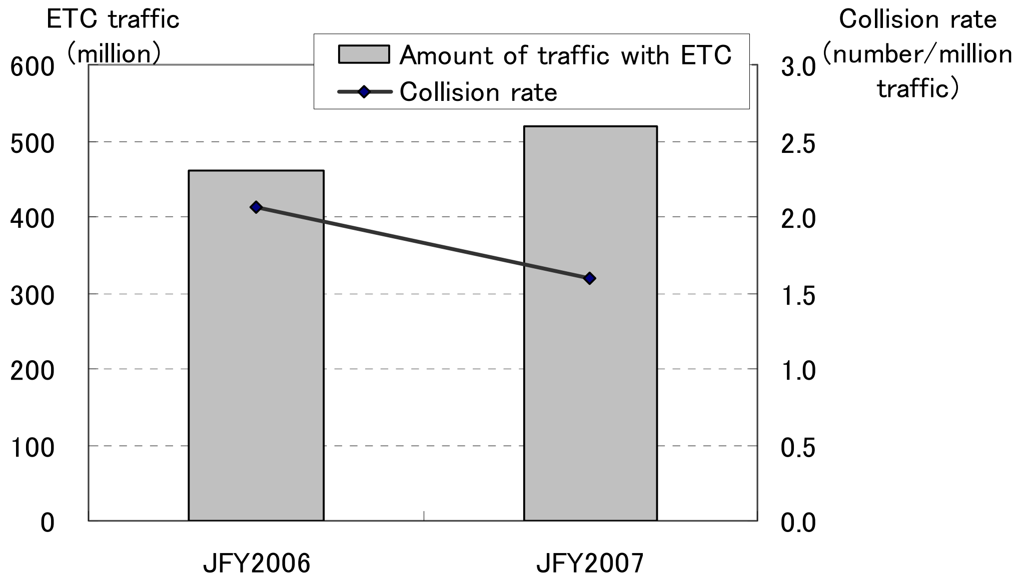



Figure 4. Collision accidents in the ETC lane
(According to NEXCO East)

### 3.2. Occurrence of the ETC system stop

In 2008, the ETC system stopped and the toll bars did not open for one out of approximately every 580 vehicles. Investigation of the causes showed that over 60% of the accidents resulted from ETC-card-related problems (no insertion of the card, improper card insertion, expired card, and the like) and over 30% resulted from vehicles without an ETC device entering into the ETC lane by mistake.

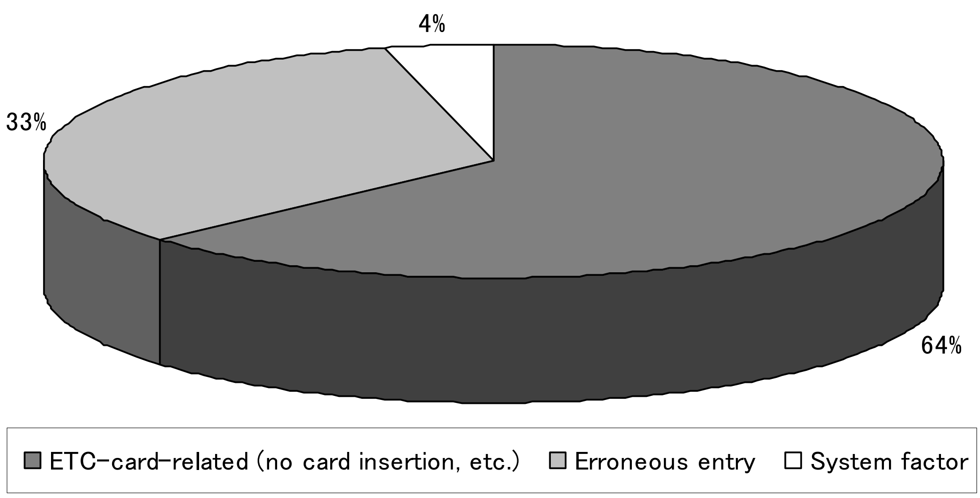



Figure 5. Causes of ETC system stop
(According to NEXCO East in JFY2007)

### 3.3. The speed in the ETC lane (focused on collision accidents)

Investigation of the collision accidents in the ETC lane revealed that over 90% of the drivers entered the ETC lane at a speed of over 20 kilometres per hour. Thus, it is assumable that the speed makes it difficult for the drivers to stop safely.

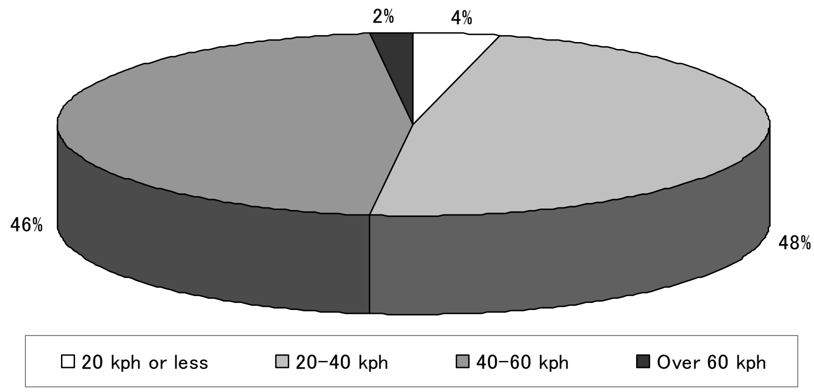



Figure 6. Speed distribution in the ETC lane when collision accident occurred
(According to NEXCO East in JFY2007)

### 3.4. The mechanism of the occurrence of collision accidents

Next, the causes of collision accidents will be reviewed. While collision accidents will not occur without objects to collide with, even when such objects are present, accidents can be avoided by avoiding contact with the objects. However, it is extremely difficult to completely eliminate these objects, which can be incidentally brought about by human-made errors, such as improper insertion or no insertion of the ETC card into the ETC device and erroneous entry of non-ETC vehicles into the ETC lane. Meanwhile, it is possible to avoid contact with the objects through drivers' habitual compliance with safety speed when passing through the ETC lane. NEXCO East has so far taken measures in terms of these two points, and the purpose of the measures is to avoid contact with objects to collide with.

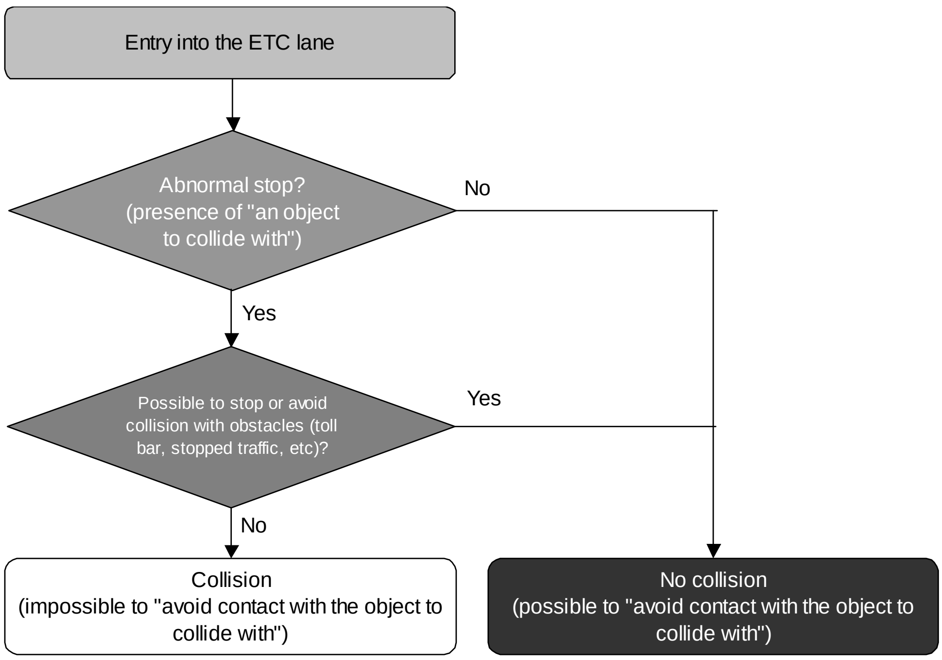



Figure 7. Mechanism of the occurrence of collision

## 4. CONVENTIONAL MEASURES FOR SLOWING DOWN THE SPEED IN THE ETC LANE

In the past, the company carried out several measures (including trials) for slowing down the speed in the ETC lane. While the measures were aimed at drawing the attention of drivers aurally, tactually and visually, none of the measures provided sufficient or continued effect.

## 5. STEPPED-UP MEASURES FOR SLOWING DOWN THE SPEED IN THE ETC LANE

### 5.1. Concept of the measures introduced

As described in the above, based on the conventional measures for slowing down the speed in the ETC lane, “warning” is given to drivers. Therefore, reducing the speed is dependent on the drivers’ decision (and decency) who have received the warning. Consequently, sufficient effect has not been obtained so far.

Thus, new measures were explored from a different perspective; that is, measures that make it physically impossible to increase the speed were explored.

#### 5.1.1. Humps and corrugated pavement

Humps and corrugated pavement are installed on some of the streets to slow down the vehicle speed. If vehicles pass through these Humps or corrugated pavement at over a given speed, the vehicles may come into contact with the road surface or the steering wheel may be uncontrollable, making it difficult to drive. Because of these effects, issues including impact of the vibration on freight and safety for two-wheel vehicles need to be sufficiently considered. Since these measures have never been adopted for expressways, it is impossible to introduce them immediately. Therefore, these measures were excluded from the stepped-up measures this time.

#### 5.1.2. Delay in the timing at which the toll bars open

In the autumn of 2007, NEXCO Central started the trial and delayed the timing at which the toll bars open in the ETC lane, and it is found that the trial has been effective. Since NEXCO East’s expressways are seamlessly connected to NEXCO Central’s expressways, it was decided that the introduction of the measures should be explored.

*Japan Highway Public Corporation was privatised and divided into three NEXCOs, i.e. NEXCO East, NEXCO Central and NEXCO West in October, 2005.

### 5.2. Principle of the measures

The principle of the measures is to delay the timing at which the toll bars open in order to slow down the speed of vehicles passing through the ETC lane. The measures enable reduction in speed physically and visually.

When a vehicle passes through the vehicle detector (see Figure. 1), a signal that activates the toll bars is generated. In the conventional system, there was little time lag between the detection by the detector and generation of the toll bar activation signal. According to the stepped-up measures, by changing the timing at which

the toll bars open, the time lag was increased by approximately 0.5 to 1 second, compared with before. The time lag differs depending on the toll gate. (This is because the distance between the vehicle detector and the toll bars varies depending on the toll gate. For reference's sake, the design scheme of NEXCO East defines a minimum of 11.9 metres for the distance.).

### 5.3. Concept of the trial

After NEXCO Central, NEXCO West also initiated a trial implementation of the measures in 2007 and subsequently it has been expanding the measures throughout the company. NEXCO East examined the effectiveness of the measures first, and launched the trial in September 2008. This is to verify whether or not various factors (the type of interchange and the amount of traffic, for example), which are different from NEXCO Central's and West's trial items, could be risk factors. Particularly, since NEXCO East has many mainline toll gates (namely, places where the amount of traffic is relatively heavy and it is difficult to slow down the speed linearly), 7 places were selected as the trial sites after reviewing all the conditions.

Table 1. Trail locations and data concerning the stepped-up measures for slowing down the speed

| Prefecture | IC | Branch | IC type | Traffic (thousand/day) | ETC utilisation rate (%) | Average speed (kph) |
|---|---|---|---|---|---|---|
| Hokkaido | Shinkawa (Sasson Expwy) | Hokkaido | Plane and direct | 11 | 59 | 29 |
| Miyagi | Iwanuma (Sendai tobu) | Tohoku | Plane and T-intersection (entrance) Y-intersection (exit) | 4 | 56 | 34 (exit) |
| Tochigi | Shikanuma (Tohoku Expwy) | Kanto | Trumpet-shaped | 14 | 73 | 36 (entrance) |
| Nagano | Kosyoku (Nagano Expwy) | Kanto | Trumpet-shaped | 10 | 77 | 33 (entrance) |
| Nagano | Omi (Nagano Expwy) | Kanto | Trumpet-shaped | 2 | 77 | 35 (entrance) |
| Chiba | Kisarazu-Kaneda (Aqualine) | Kanto | Mainline toll gate | 13 | 86 | 43 (exit) |
| Niigata | Niigata West No.1 (Hokuriku Expwy) | Niigata | Mainline toll gate | 19 | 73 | 37 (exit) |

### 5.4. Public relations activities concerning the trial

In order to lower the speed of vehicles passing through the ETC lane and reduce trouble such as collision with the toll bars through the implementation of the measures, it is necessary that customers such as drivers fully understand the details and the purpose of the measures. Therefore, PR activities were carried out. The locations where the PR activities should be conducted were discussed to include various locations before driving, on the expressway, and immediately before the ETC lane. For drivers before driving, the information was aired on the radio commercial. For drivers on the expressway, the posters were put up and leaflets were distributed in rest areas, and the information was provided on variable message signs and through the expressway radio. Sign boards and LED display devices were installed, and security officers alerted drivers immediately before the ETC lane (LED display devices were installed only for the first 2 to 3 months at trial sites where the amount of traffic was relatively

heavy, and security staff alerted drivers only for the first 1 to 2 weeks at each trial site.).

Photo 1. Information on variable message sign


Photo 2. Information on LED display device


Photo 3. Security staff alerting drivers

## 5.5. Trial results

At the Hokuriku Expressway Niigata West toll gate, which is in accord with the mainline toll gate in terms of type and traffic, the trial started in 25 September 2008. It was found that the share of vehicles number passing at a relatively high speed of 40 kilometres per hour or higher has been reduced to 1/4, and the rate of number of vehicles colliding with the toll bars has been reduced to 1/2 or less. In addition, four months after the initiation of the trial, the measures have still been effective. It is thought that these successes are the result of the sufficient promotion of the measures and drivers' good understanding of the purpose of the measures. Moreover, there have been no accidents or complaints reported.

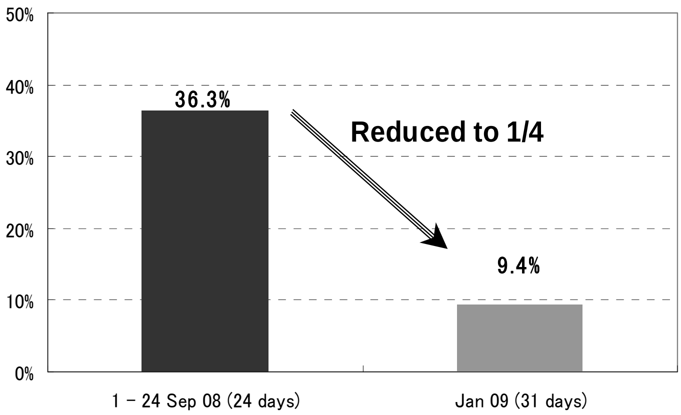



Figure 8. Transition of the share of vehicles number at 40 kph or higher at Niigata West entrance toll gate

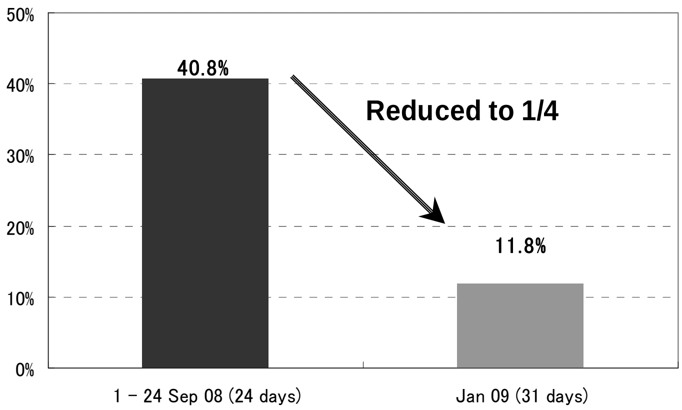



Figure 9. Transition of the share of vehicles number at 40 kph or higher at Niigata West exit toll gate

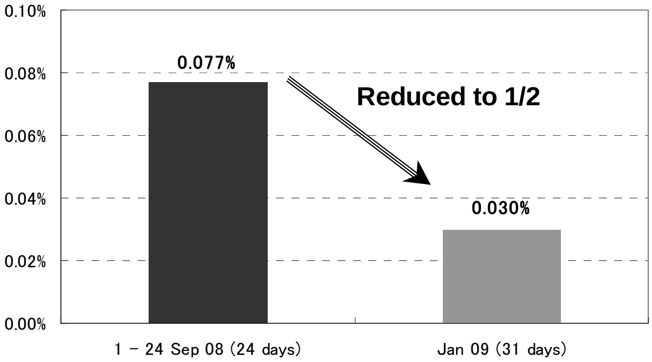



Figure 10. Transition of the rate of number of vehicles colliding to the toll bars at Niigata West entrance toll gate

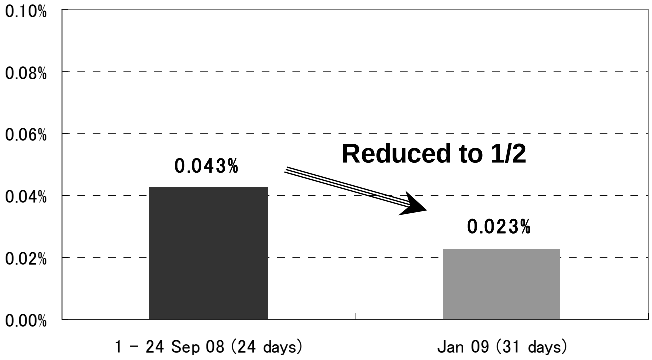



Figure 11. Transition of the rate of number of vehicles colliding to the toll bars at Niigata West exit toll gate

### 5.6. Future issues (for company-wide operation)

From these trial results, the effectiveness of the measures (not only reduction of the speed, but also reduction of the number of vehicles colliding with the toll bars) was confirmed. In addition, since no problems, such as receiving complaints, have been confirmed, NEXCO East decided to expand the measures throughout the company.

It is thought that the success of the trial is due to the sufficient promotion of the measures. Thus, for the company-wide operation, the measures need to be promoted through even more effective PR activities than those of the trial, to gain customers' (drivers') understanding of the measures.

## 6. THE MEASURE FOR RESTRAINING THE SPEED DISPERSION IN THE ETC LANE (DISPLAYING THE TIMING AT WHICH THE TOLL BARS OPEN)

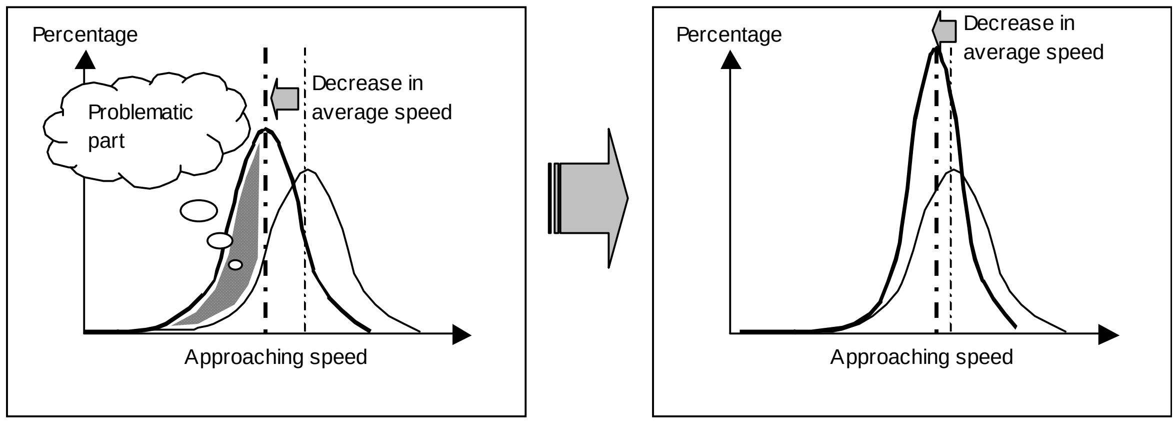
### 6.1. Background and purpose

The data from the trial previously conducted by NEXCO Central showed that the measures caused the overall speed distribution in the ETC lane to shift to a lower speed range. Namely, it was found that not only drivers who normally passed at a relatively high speed, but also drivers who normally passed at a relatively low speed are passing at an even lower speed than before. For this reason, the following issues are assumed.

(1) Even drivers who normally passed through the ETC lane at a specified speed are influenced by the traffic flow or the like and put on the brake, which may cause stress to the drivers.

(2) In order to achieve a smooth flow of traffic, the speed distribution needs be as small as possible. According to the trial results by NEXCO Central, there is a possibility that the measures widened the speed distribution.

Therefore, in order to ensure smaller speed distribution and smoother traffic flow, as a trial, drivers were visually notified of the timing at which the toll bars opened.




Change of speed distribution after the implementation of the slowing down measures

Desirable change of speed distribution

Before the implementation

After the implementation

Figure 12. Change of speed distribution

## 6.2. The principle of the measures

In view of the existing equipment and the like, a general-purpose LED display device was installed near the toll bars. The timing at which the toll bars opened was displayed by providing information in a countdown manner.

The vehicle detector (Fig. 1) located most upstream of the ETC lane was used as a trigger to activate the LED display device, as in the stepped-up measures for slowing down the speed. Therefore, since the distance between the vehicle detector and the LED display device was not sufficiently long (merely more than 10 metres), multi-step change of information displayed on the LED display device was made within a short time.

Photo 4. Installation of LED display device

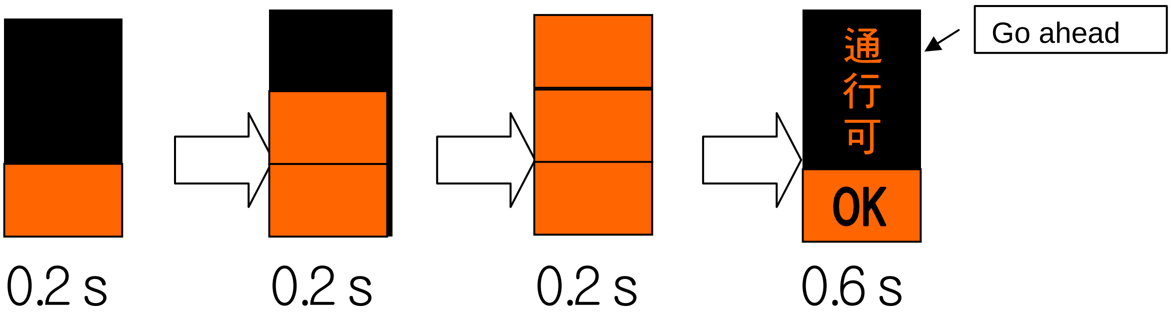



Figure 13. Information displayed and switching time of LED display device (“s” for second)

## 6.3. Preparation for the trial

In order to evaluate the effectiveness of the trial using the LED display device in the reduction of the speed dispersion, whether the presence of the LED display device affects the speed dispersion needs to be examined. If only some of the multiple ETC lanes, which are arranged next to each other in the same direction and are located in the same toll gate, have the LED display device, there is a possibility that drivers may be confused about which lane to enter. Therefore, by taking into account the direction at one toll gate, it was decided that the LED display devices should be installed at either the entrance or the exit, and the trail was then conducted.

When selecting the toll gate to be provided with LED display devices, one having approximately the same traffic amount and speed distribution at its entrance and exit was sought. In view of this, the Hokuriku Expressway Niigata West toll gate was selected as the trial site, which was one of the trial sites at which the measures for slowing down the speed were implemented (unlike a typical type of IC with curvy ramps, the Niigata West toll gate is located at a place that linearly and directly connects to a national highway from the expressway mainline, and the gate is similar to a mainline toll gate.).

As to whether the entrance or the exit should be chosen, several factors were examined. Since the hourly

traffic volume was determined as a main factor, the trial was implemented at the entrance (rapid increase in hourly traffic volume might increase the risk of rear-end collision.).

## 6.4. Results of the trial

### 6.4.1 Distribution of approaching speed into the ETC lane

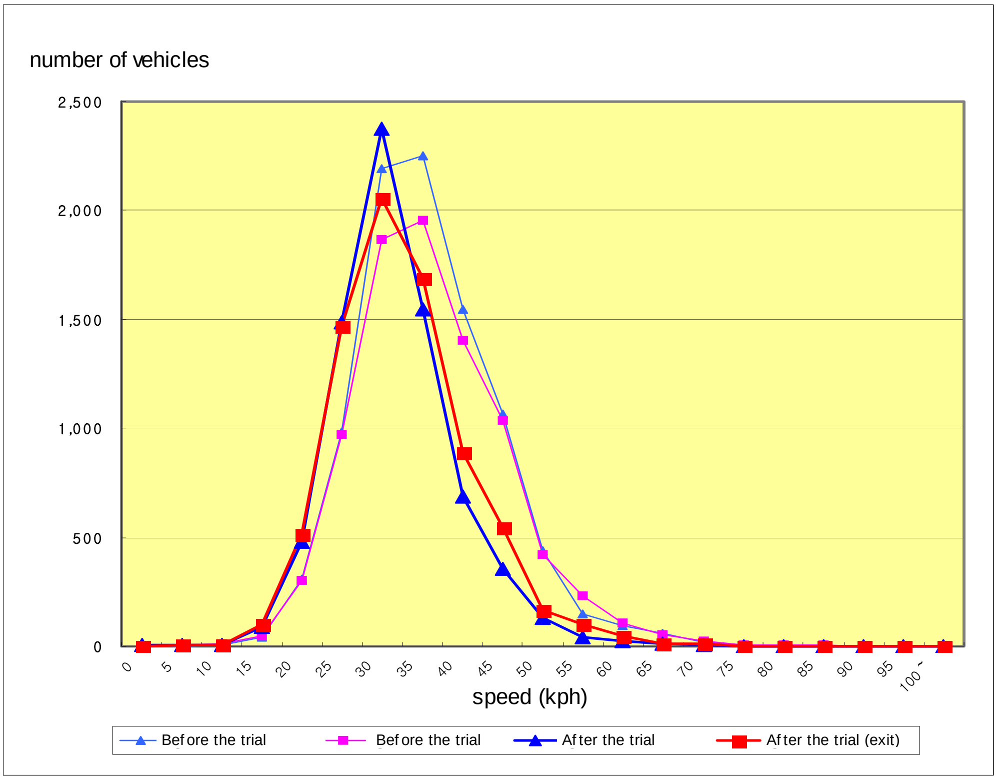



Figure 14. Speed distribution in the ETC lane at Niigata West toll gate
(before and after the implementation of the trial for slowing down the speed)

The distribution of approaching speed into the ETC lane shows no significant differences between the exit at which the trial for slowing down the speed was implemented and the entrance at which the trial was not implemented. However, since a decrease in the high speed range was seen at the exit, it can be assumed that the result is attributable to the effects of the display of the timing at which the toll bars open.

### 6.4.2 Results of the questionnaire survey

According to the results of the questionnaire survey conducted at a rest area located adjacent to the Niigata West toll gate, approximately 70% of the drivers answered that they noticed the display of the timing and found it useful. There were other drivers, stating “It was not easy to find the LED display,” “The display time was short,” and “A blinking or multiple-colour LED display could be better.” Since the LED display device had to be installed between existing equipment, the device could not be installed at the best place for the customers.

However, the trail was conducted smoothly without any accidents or complaints.

## 6.5. Future issues

Further effective methods using the display of the timing are being explored. As to the installation position of the LED display device, visibility from customers needs to be the first priority. Other issues include an extension of the display time of the LED display device, change of the display design, change of the number of display patterns (the number of display changes), and use of colours.

# 7. CONCLUSION

NEXCO East implemented the company-wide stepped-up measures for slowing down the speed on 16 March 2009 and had finished by the end of this April, after the examination of the effectiveness of the measures, selection of the trial sites, implementation of the trails, and assessment of the trial results. There have been no accidents so far. The measures are straightforward but found to be very effective, and thus, the company will continuously focus on the stability of the effects and the operation of the measures.

In addition, as to the effects of the display of the timing, the presence of the display device has so far indicated no significant difference in speed distribution. Therefore, the company needs to make improvements in providing customers with information, including display methods and contents to be displayed.

In closing, the deepest appreciation goes to NEXCO Central for suggesting the introduction of the measures for slowing down the speed to NEXCO East.

# REFERENCES

1. ETC Handbook 2008 edition, ORSE: Organization for Road System Enhancement

# STUDIES ON TRAVEL TIME RELIABILITY FOR INTERCITY EXPRESSWAY UNDER VARIOUS EVENTS

**Takeshi Mastsushita**

*Traffic Division Maintenance and Operation Department,*

*West Nippon Expressway Company Ltd.*

*Dojima Avanza 19F, 1-6-20 Dojima, Kita-ku, Osaka, 530-0003, Japan*

*t.matsushita.ab@w-nexco.co.jp*

## ABSTRACT

NEXCO-West, one of the intercity toll road operators in Japan, provides travel time information on its expressways. Currently, travel time information is provided in limited areas, and those information has less priority in case of traffic accident. travel time. Moreover, many signboards only provide information on congestion length, during the congestion caused by traffic accidents.

As a result, many customers complain about lack of travel time information in case of traffic jams and traffic accidents, since they spend more time than they have expected.

Using the traffic speed data measured by vehicle sensors near the major traffic congestion points, this paper describes results of analysis on distribution of travel time during various events occurred in the past year. The paper also reviews travel time an indicator called Buffer Time (*BT*), and suggests more comprehensive travel time indices to drivers compared to *BT*.

**KEY WORDS**

BT, travel time reliability

## 1. INTRODUCTION:

West Nippon Expressway Company (hereafter “NEXCO-West”) is operating and managing expressways over 3,300km in length in the western part of Japan.
Approximately 2.3 million traffics are on its expressways every day. (Apr-2009)
We are providing real-time traffic information by various means such as Variable Message Signboard (hereafter “VMS”) at highway entrance and exit, , toll gates, or JCTs, and radio broadcasting. Furthermore, we are providing the same information by use of cellular phone using internet function.
It is true that real-time information is informative and helpful for driver’s who are about to leave their home or office, but these data cannot be useful for driver’s decision making on their travel plan for his future trip (i.e., tomorrow or maybe next week).
We are providing information of congestion forecast, and estimated travel time to destination IC for 5 months by use of the record of past 3 years at areas in heavy inbound traffic congestion.
Therefore, these data are not useful once the incidents occurred.
Especially, people who need to catch international flight must be very punctual, and travel time reliability is very important. However, so far the travel time information has not been enough to meet their demand.
In this paper, results of analyses concerning travel time reliability is described, by calculating the required time for any departure time with or without incident.
Some case studies were performed using sample sections of 30km to 100km interval including the areas with frequent traffic congestion.

## 2. METHOD OF ANALYSIS

### 2.1 Target of analysis

For our case-study, 5 sections shown in Table 1 was used as sample cases. They were selected based on their frequent occurrence of traffic congestion between 2008-Jan-1 to 2008-Dec-31 (365days).
In this paper, results of case studies for section #3 and #4 is described.

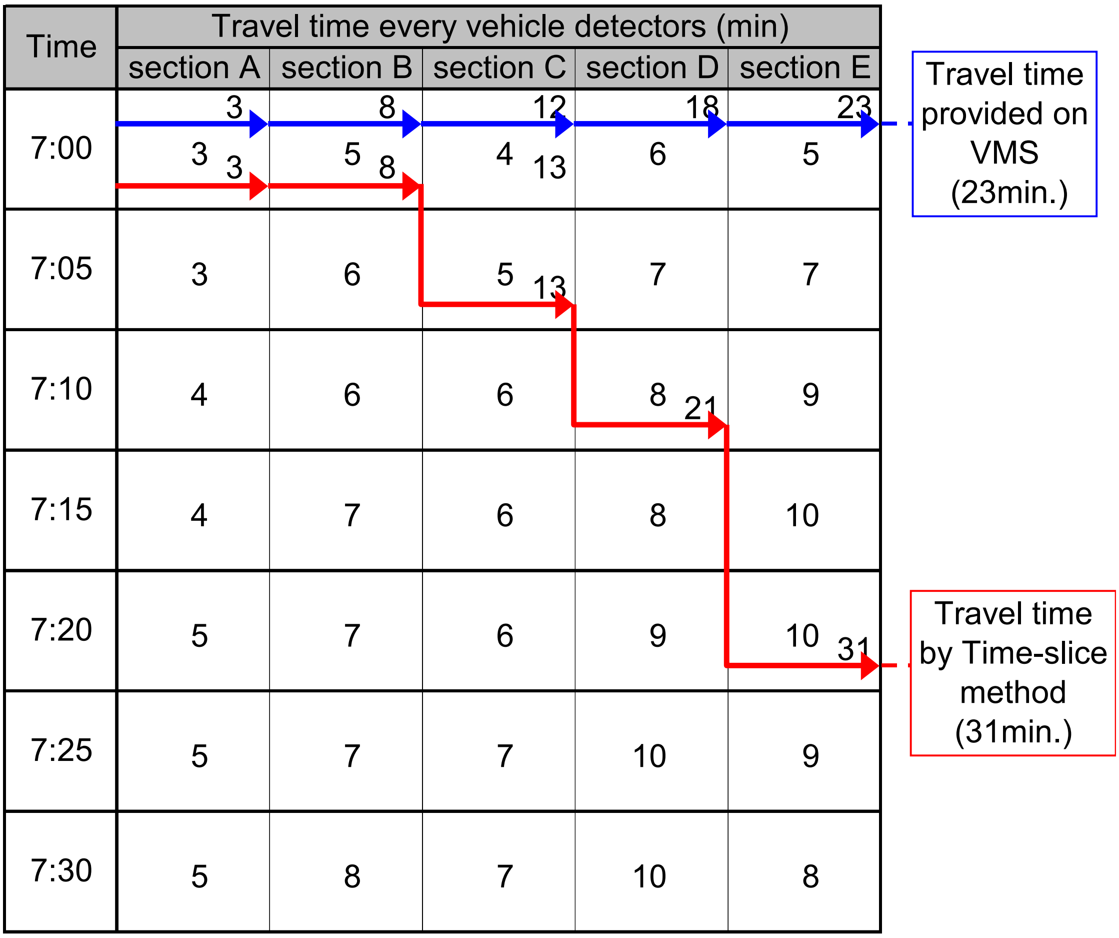
Table 1 Target of analysis

| № | Road Name | Section | | | Length | Congestion Number of Times (2007) |
|---|---|---|---|---|---|---|
| 1 | Meishin-EXPWY | Suita | ～ | Nishinomiya | 21.3 km | 174 |
| 2 | Kinki-EXPWY | Suita | ～ | Matsubara | 27.5 km | 1551 |
| 3 | Chugoku-EXPWY | Chugoku-Suita | ～ | Nishinomiya-kita | 28.9 km | 769 |
| 4 | Hanwa-EXPWY | Matsubara | ～ | Gobo | 101.9 km | 660 |
| 5 | Nishimeihan-EXPWY | Matsubara | ～ | Tenri | 27.2 km | 465 |

### 2.2 Calculating travel time by time-slice method

Time-slice method a is a dynamic travel time estimation method to calculate travel time by considering elapsed time from departure and dynamically summing up each travel time.
The velocity data were provided from vehicle detectors located every 2km interval.
In this paper, departure time was set every 5 minutes, requiring 12 samples within 1 hour.
Travel time provided on VMS is the estimation at the time of departure.

Fig. 1 Conception of Time-slice method




**2.3 The classification of travel time**
We analyzed the decrease of travel time reliability at the time of incidents (i.e., traffic accidents or road work) by classifying calculated travel time with or without incident using the past record of congestion data.

## 3. TRAVEL TIME RELIABILITY ANALYSIS BY THE DAY

We calculated a reliability index by the days of the week with a normal time reliability index without the incident to analyze the change of the travel time reliability by the difference of the day.

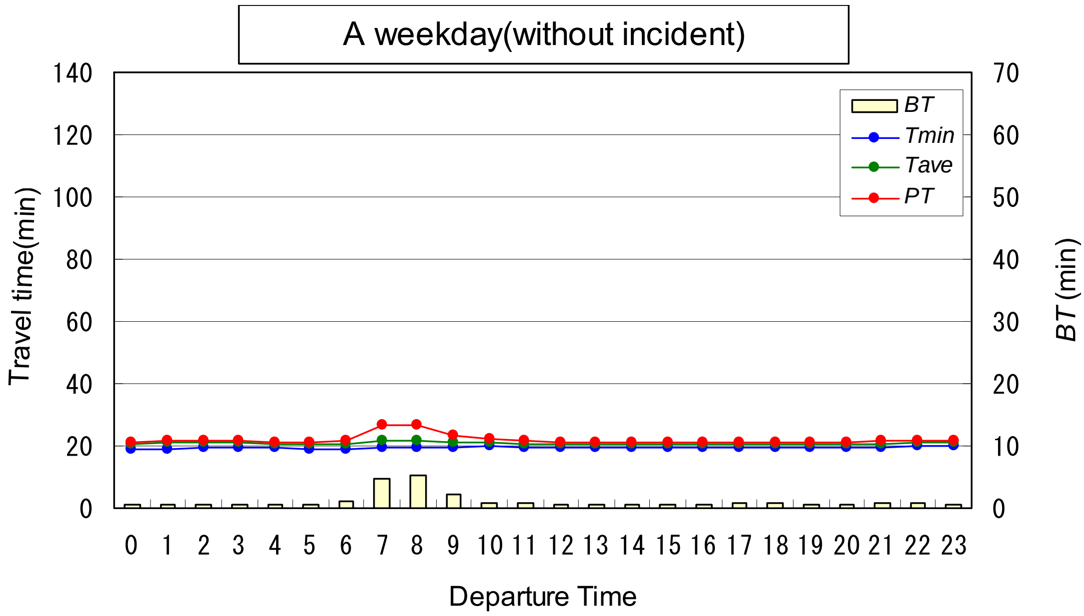
### 3.1 Section #3 Chugoku Expressway Chugoku-Suita – Nishinomiya-kita (30km)

This section is experiencing frequent congestion in almost whole section on weekend and seasonal traffic congestion period.

#### 3.1.1 A weekday

Travel time is increased from 7:00 am to 9:00 am, and the peak point is shown around 8:00 am.
Around the peak period, Average travel time (*Tave*) is 21 minutes (2 more minutes compared to the 5%-tile travel time (*Tmin*) of 19minutes).
95%-tile travel time (Planning Time hereafter *PT*) was about 27 minutes (8 minutes increase from *Tmin*) Buffer time (*BT*) was 5 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 3 minutes for each time. *BT* for each hour is almost limited within one minute, (See Fig-2)

Fig. 2 Travel time ( a weekday) without incident

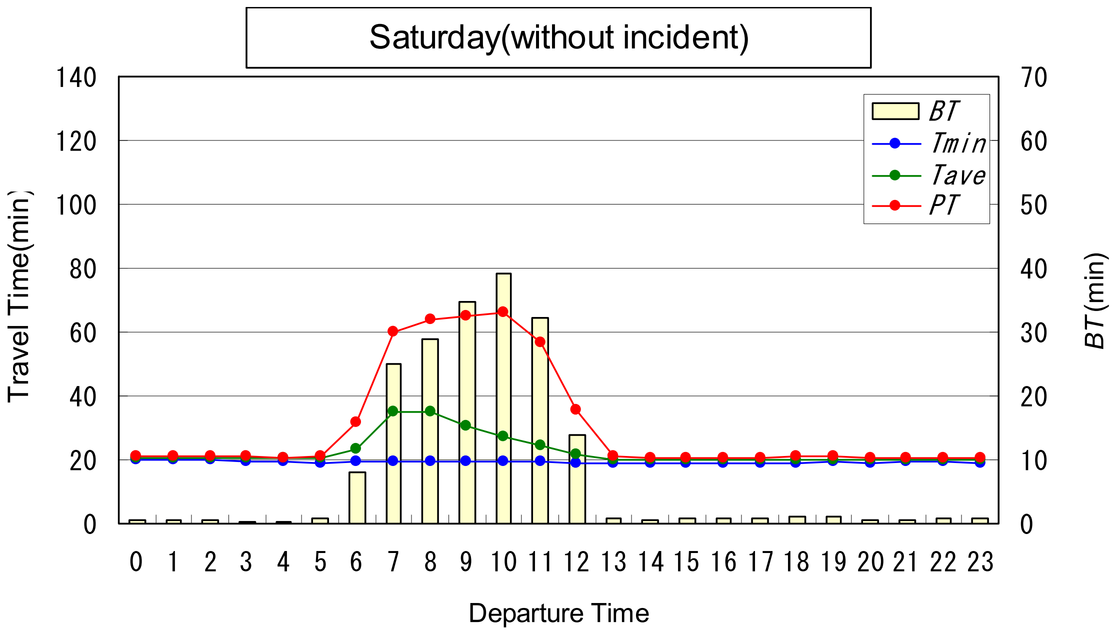



**3.1.2 Saturday**

Travel time is increased from 6:00 am to 12:00 am, and the peak point is shown around 10:00 am. Around the peak period, Average travel time (*Tave*) is 27 minutes (7 more minutes compared to the 5%-tile travel time (*Tmin*) of 20minutes). *PT* was about 66 minutes (46 minutes increase from *Tmin*). Buffer time (*BT*) was 39 minutes.

The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 2 minutes for each time. *BT* for each hour is almost limited within one minute, (See Fig-3)

Fig. 3 Travel time (Saturday) without incident

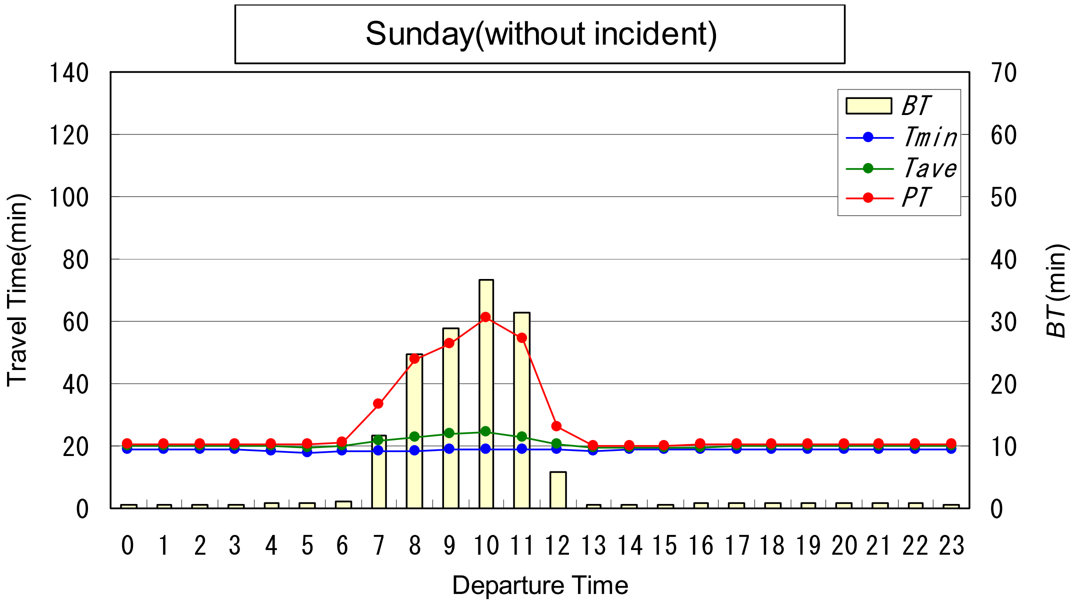



**3.1.3 Sunday / Holiday**

Travel time is increased from 7:00 am to 12:00 am, and the peak point is shown around 10:00 am. Around the peak period, Average travel time (*Tave*) is 24 minutes (5 more minutes compared to the 5%-tile travel time (*Tmin*) of 19minutes). *PT* was about 61 minutes (42 minutes increase from *Tmin*). Buffer time (*BT*) was 37 minutes.

The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 3 minutes for each time. *BT* for each hour is almost limited within one minute, (See Fig-4)

Fig. 4 Travel time (Sunday / Horiday) without incident

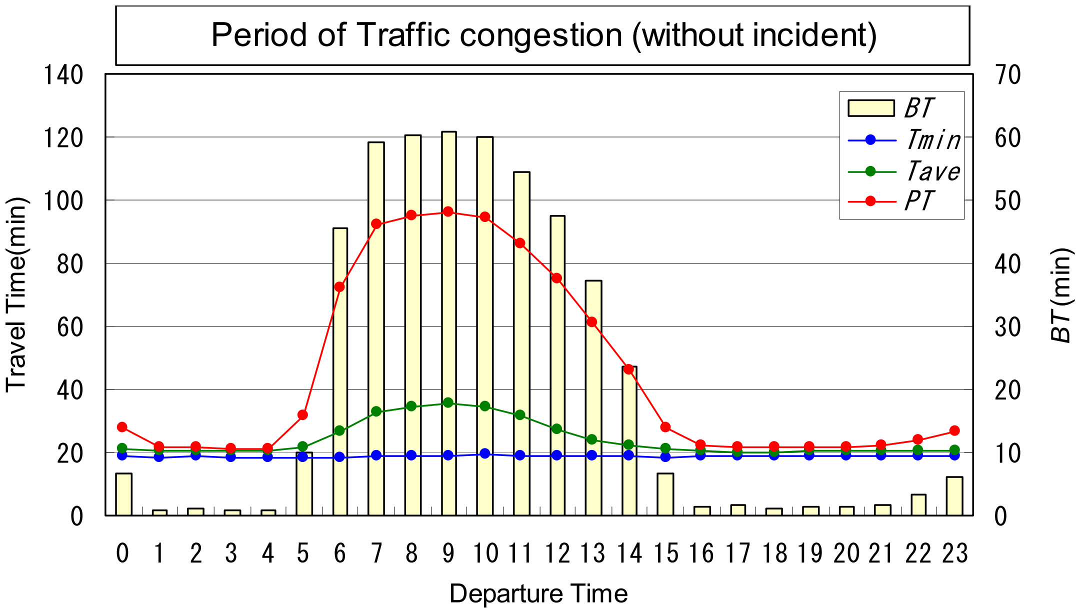



**3.1.4 Period of Traffic congestion**

Travel time is increased from 5:00 am to 3:00 pm, and the peak point is shown around 9:00 am. Around the peak period, Average travel time (*Tave*) is 35 minutes (16 more minutes compared to the 5%-tile travel time (*Tmin*) of 19minutes). *PT* was about 96 minutes (77 minutes increase from *Tmin*). Buffer time (*BT*) was 61 minutes.

The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 3 minutes for each time. *BT* for each hour is almost limited within 2 minutes, (See Fig-5)

Fig. 5 Travel time (Period of Traffic congestion) without incident



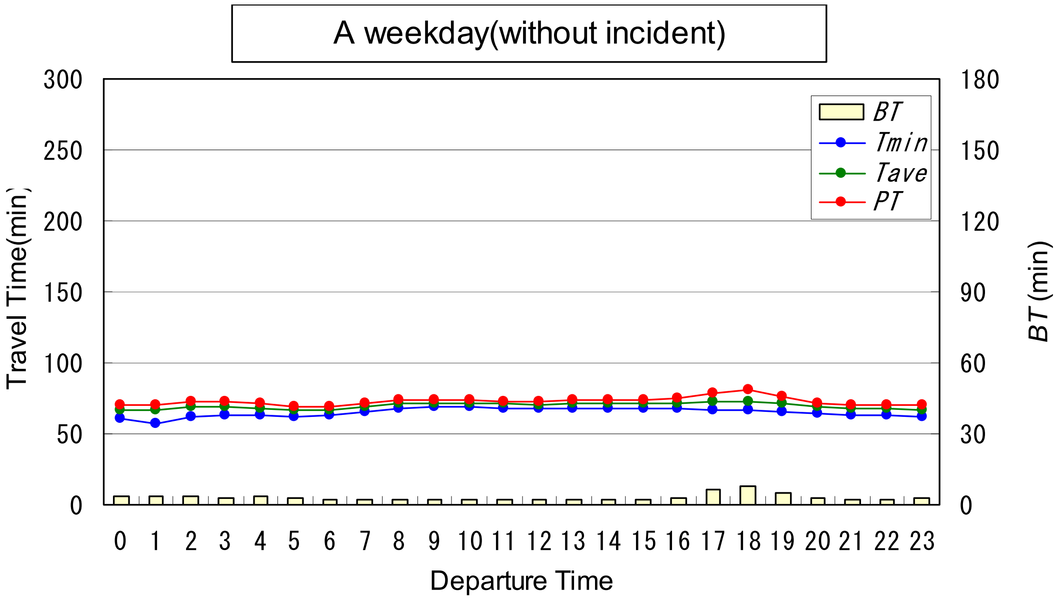
### 3.2 Section #4 Hanwa Expressway Gobo – Matsubara (100km)

This section is experiencing frequent congestion about 20 km on weekend and seasonal traffic congestion period.

#### 3.2.1 A weekday

In comparison with other days, a change is small for weekday travel time. (See Fig-6)

Fig. 6 Travel time (A weekday) without incident



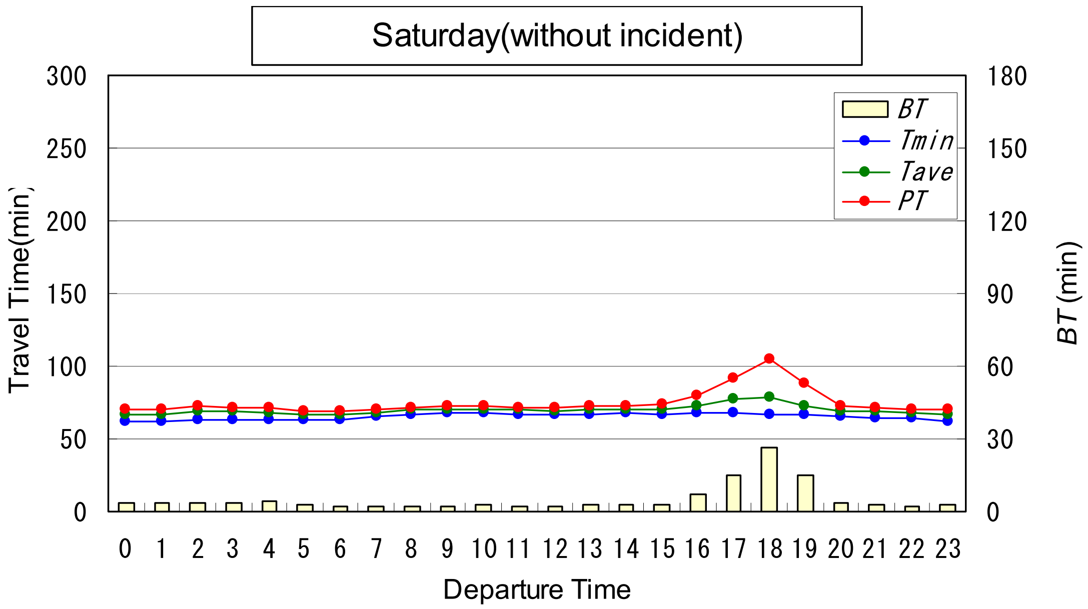

#### 3.2.2 Saturday

Travel time is increased from 4:00 pm to 7:00 pm, and the peak point is shown around 6:00 pm.

Around the peak period, Average travel time (*Tave*) is 78 minutes (11 more minutes compared to the 5%-tile travel time (*Tmin*) of 67minutes). *PT* was about 105 minutes (38 minutes increase from *Tmin*). Buffer time (*BT*) was 27 minutes.

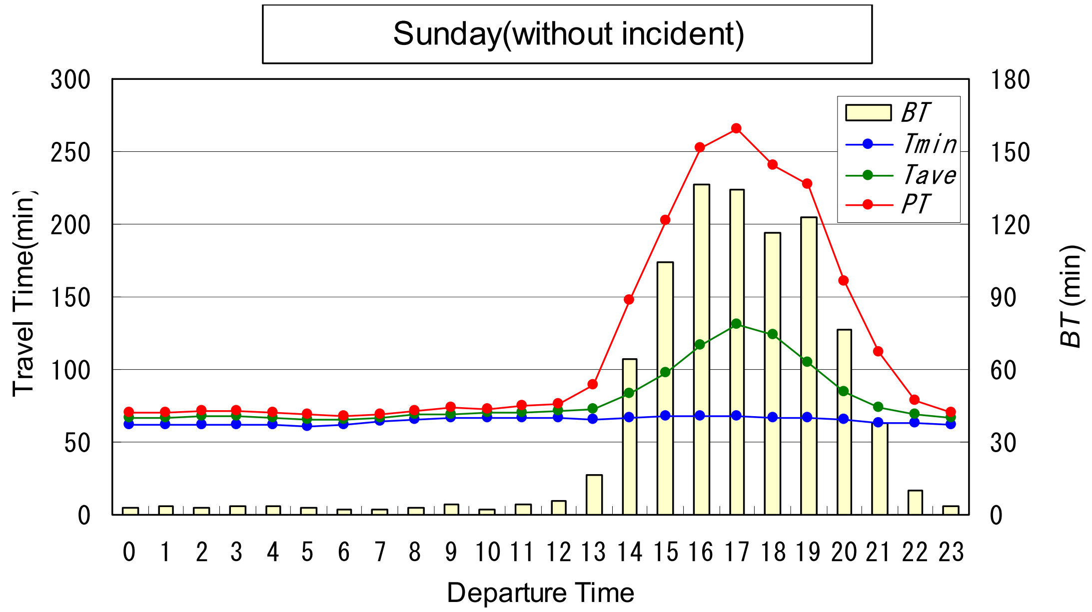
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 7 minutes for each time. *BT* for each hour is almost limited within 3 minutes, (See Fig-7)

Fig. 7 Travel time (Saturday) without incident



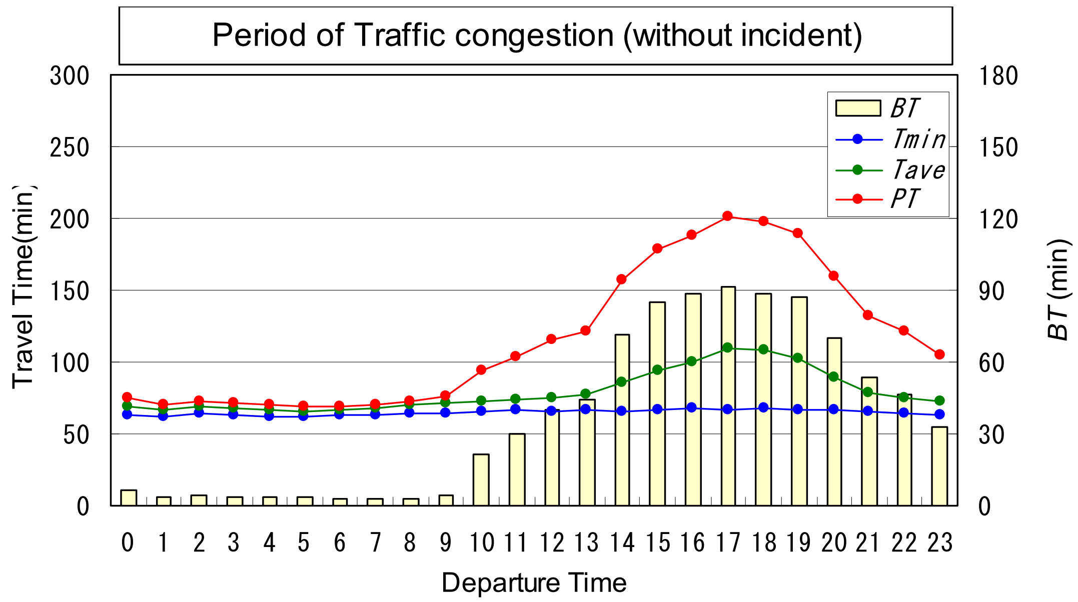
**3.2.3 Sunday / Holiday**
Travel time is increased from 12:00 am to 10:00 pm, and the peak point is shown around 5:00 pm.
Around the peak period, Average travel time (*Tave*) is 131 minutes (64 more minutes compared to the 5%-tile travel time (*Tmin*) of 67minutes). *PT* was about 265 minutes (198 minutes increase from *Tmin*). Buffer time (*BT*) was 134 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 8 minutes for each time. *BT* for each hour is almost limited within 5 minutes, (See Fig-8)

Fig. 8 Travel time (Sunday / Horiday) without incident




**3.2.4 Period of Traffic congestion**
Travel time is increased from 9:00 am to 1:00 am, and the peak point is shown around 5:00 pm.
Around the peak period, Average travel time (*Tave*) is 110 minutes (43 more minutes compared to the 5%-tile travel time (*Tmin*) of 67minutes). *PT* was about 201 minutes (134 minutes increase from *Tmin*). Buffer time (*BT*) was 92 minutes.
The change of the travel time is small except above mentioned time, and the difference of *Tmin* and *PT* is less than 9 minutes for each time. *BT* for each hour is almost limited within 4 minutes, (See Fig-9)

Fig. 9 Travel time (Period of Traffic congestion) without incident



## 4. TRAVEL TIME RELIABILITY ACCORDING TO THE INCIDENT PRESENCE

In order to figure out the deteriorating condition of travel time reliability with incidents, we compared Planning Time (*PT*) at normal time (without incident) and the case with incident. (See Table-2)
In addition, 2 sections above mentioned, we compared travel time reliability according to incident presence. (See Fig-10,11)

Table 2. Travel Time reliability according to the incident presence

| № | Road Name | Section | | | Length | A Day | Departure Time | *PT* (min) | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | Normal (A) | Incident (B) | (B)-(A) |
| 1 | Meishin-EXPWY | Suita | ～ | Nishinomiya | 21.3 km | Period of Traffic congestion | 11:00am | 25.9 | 87.1 | 61.2 |
| | | | | | | Weekday | 9:00am | 15.7 | 64.2 | 48.5 |
| 2 | Kinki-EXPWY | Suita | ～ | Matsubara | 27.5 km | Saturday | 8:00am | 26.9 | 119.2 | 92.3 |
| | | | | | | Period of Traffic congestion | 8:00am | 31.3 | 145.8 | 114.5 |
| 3 | Chugoku-EXPWY | Chugoku-Suita | ～ | Nishinomiya - kita | 28.9 km | Weekday | 4:00pm | 22.7 | 118.3 | 95.6 |
| | | | | | | Period of Traffic congestion | 1:00pm | 60.9 | 110.0 | 49.1 |
| 4 | Hanwa-EXPWY | Matsubara | ～ | Gobo | 102 km | Sunday/Horiday | 5:00pm | 265.2 | 381.5 | 116.3 |
| | | | | | | Saturday | 8:00am | 98.9 | 326.6 | 227.7 |
| 5 | Nishimeihan-EXPWY | Matsubara | ～ | Tenri | 27.2 km | Weekday | 6:00pm | 20.7 | 132.7 | 112.1 |
| | | | | | | Weekday | 8:00am | 19.9 | 128.9 | 109.0 |

Fig. 10 Travel time (Period of Traffic congestion) without incident

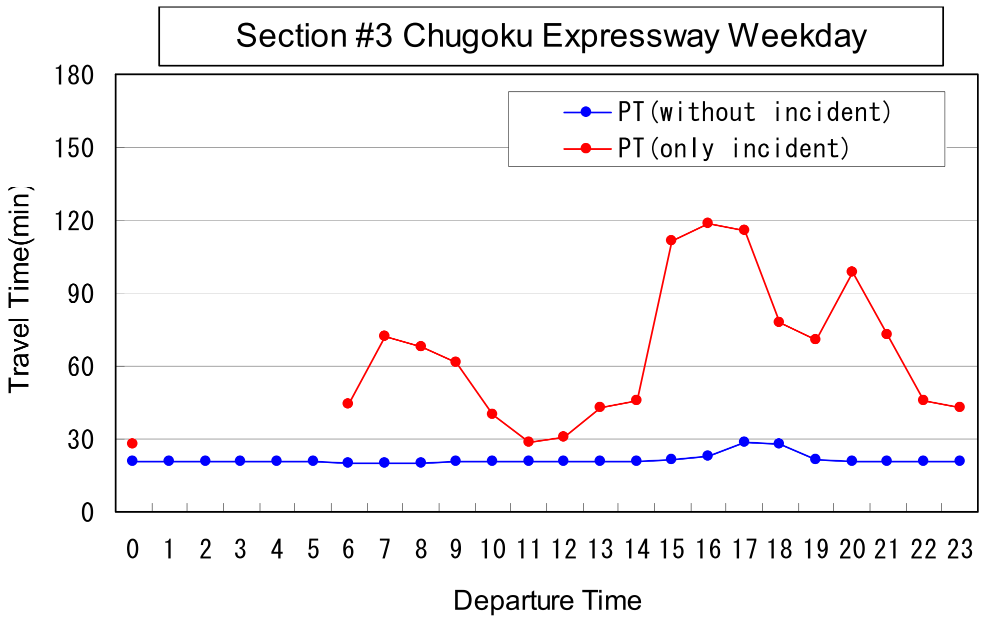
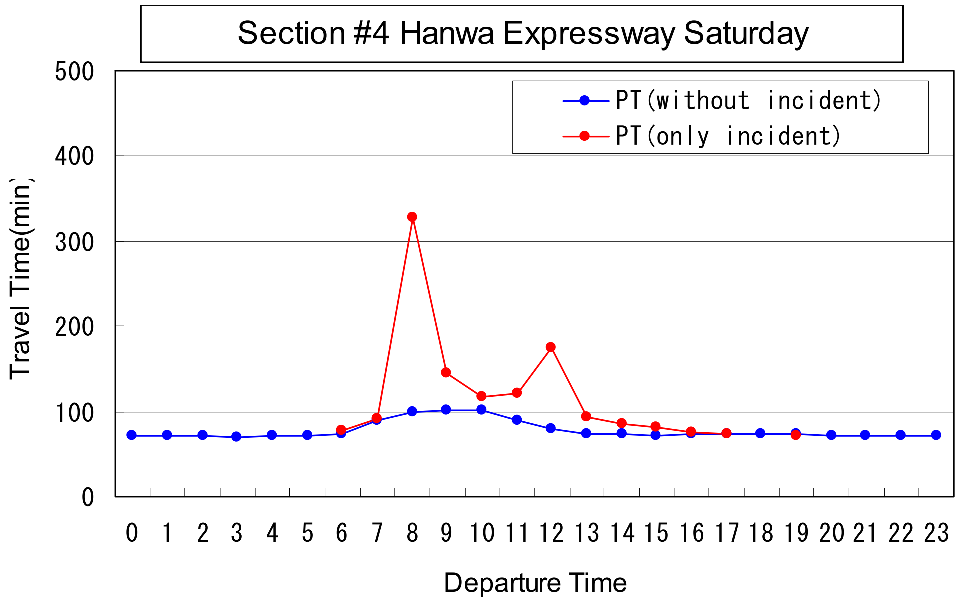


Fig. 11 Travel time (Period of Traffic congestion) without incident




*PT* with incident is larger than normal time each sections and each day. If many drivers are caught in a jam caused by incident, they are usually forced to spend more travel time than expected.
Especially, maximum value of *PT* with incident concerning the section No.4 (Hanwa Expwy from Matsubara to Gobo) is 327 minutes, drivers are forced unexpected travel time for 228 minutes in comparison with *PT* at normal time (99 minutes).
Maximum value of *PT* with incident is marked on various days, and the characteristic of day is almost negligible, the phenomenon may be caused anytime.
However, probability of encountering incident will be at most 10% in the section we analyzed, Whether the risk is acceptable or not depends on the drivers' perspective.

## 5. SUMMARY AND CONCLUSION

Currently, travel time reliability is not familiar with highway uses.
In order to establish a travel time estimation system like the time schedule of the railway, we must investigate users' acceptable range of the reliability. For example, we have to ask the truck drivers' perspective of the Buffer Time (95%-tile travel time or 80%-tile travel time or others) in order to minimize their risks.
At the same time, we must consider how we could advertise to the public and let them get these data easily. For example, we can recommend drivers the departure time at prearranged accommodation by working together with lodging reservation website.
In this study, the analysis was performed with the past 1 year data. We have to improve reliability by analyzing more data.

# *ASSESSING TRAVELLERS' PERCEPTIONS for ACHIEVING SUSTAINABLE TRANSPORTATION: A CASE STUDY in KLANG VALLEY*

**Jen Sim HO[1], Lee Vien LEONG[2], and Ahmad Farhan MOHD SADULLAH[3]**
*School of Civil Engineering, Universiti Sains Malaysia, Engineering Campus,*
*14300 Nibong Tebal, Seberang Perai Selatan,*
*Pulau Pinang, Malaysia*
*jensimho@yahoo.com.my*

**ABSTRACT**
In Malaysia, alleviating traffic congestion in major cities has always been given the top priority. Despite various remedial measures and efforts to improve public transport in Kuala Lumpur, there are nearly 2.2 million private vehicles moving into the city centre daily. A survey involving 700 motorcyclists and car users was conducted in Klang Valley to understand their perception on public transport services as well as to gauge their willingness to switch to public transport. It was established that in general, the public transport services were poorly rated. However, motorists have shown some interests to switch to public transport if incentives were offered. Factors such as comfort and cleanliness, personal status and privacy, stress, safety, sending kids to school and irregular of working hours were found to be significantly different between car users and motorcyclists. A discrete choice model was also developed to explore the social economic factors influencing the use of private vehicles.

## 1. INTRODUCTION

The number of vehicles in Malaysia has been seen to have increased tremendously at the rate of 8% annually over the past few decades without sign of slowing down. Of all the total number of vehicle ownerships, it was estimated that more than 25% are operated in Klang Valley alone (Mohamad, 2003). Table 1 shows the statistics of vehicles registered in Malaysia from 1996 to 2007. It is also recorded that motorcycles constitute about 49% of the total vehicles and at the same time, 59% (6378 cases) of the total numbers of deaths resulting from road accidents involved motorcyclists and pillion riders were reported in year 2006 alone (Johari et al, 2007).

Table 1 Statistics of New Vehicles Registration in Malaysia Between 1996 – 2007 (Source: Road Transport Department)

| Year | Motorcycle | Car | Bus | Taxi | Hire & Drive Car | Goods Vehicle | Others |
|---|---|---|---|---|---|---|---|
| 1996 | 322145 | 318765 | 2620 | 4358 | 2545 | 69234 | 30844 |
| 1997 | 364214 | 372343 | 2947 | 5257 | 1800 | 65160 | 28396 |
| 1998 | 237776 | 159642 | 797 | 3569 | 552 | 11786 | 6342 |
| 1999 | 236779 | 296716 | 506 | 1925 | 1724 | 19987 | 8102 |
| 2000 | 238695 | 344847 | 544 | 2635 | 2883 | 24316 | 11949 |
| 2001 | 234751 | 395591 | 652 | 3169 | 1348 | 25512 | 13866 |
| 2002 | 222685 | 419713 | 919 | 4446 | 1242 | 25415 | 16768 |
| 2003 | 321234 | 424753 | 1014 | 5542 | 1232 | 29975 | 17041 |
| 2004 | 397977 | 472116 | 1290 | 7746 | 1797 | 33169 | 18268 |
| 2005 | 422255 | 537900 | 1568 | 5002 | 3411 | 33532 | 16440 |
| 2006 | 448,806 | 458,293 | 2,023 | 4,808 | 2,414 | 35,974 | 22,142 |
| 2007 | 484,394 | 419,449 | 1,786 | 3,803 | 2,478 | 40,954 | 6,850 |

Given the high accident rates involving motorcyclist and pillion riders, various efforts were devoted to study and mitigate the motorcycle related accidents. It is expected that the motorcycle ownerships would continue to rise owing to some contributing factors especially

ease of manoeuvre due to its small size, consequence of traffic congestion due to poor public transportation system, plenty of parking spaces, and burden of high petrol price (Leong and Sadullah, 2007).

According to a study initiated by Syarikat Prasarana Nasional Bhd (SPNB) (The Star, 2006), there were 2.2 million private vehicles moving into the city daily. The present of motorcycles in mixed traffic flow during traffic congestion is extremely dangerous as these motorcycles often weave in and out between the queuing vehicles to speed up their journey. In addition, the excessive influx of private vehicles into Klang Valley has triggered concerns on the associated adverse impacts such as traffic congestion and air pollution that creates considerable pressure on the road network systems. Many decision makers and researchers agreed that this situation is not sustainable. Therefore, this study is to identify the underlying factors affecting the private vehicle use especially the most risky mode of transportation which is motorcycle.

## 2. METHODOLOGY

This study was conducted in Klang Valley in May 2007 through questionnaires. The target respondents were car users and motorcyclists.

A total of 700 samples collected. Of which, about 10% of the questionnaires were excluded due to incompleteness. The reasons for incompleteness of the survey forms include the reluctance of respondents to answer several personal questions and the limited time they can spend to answer the survey. Of the remaining samples (639 samples), 24% were motorcyclists.

The questionnaire comprises of three sections. In the first section, the personal background data were sought. The second section was designed to obtain their travel characteristics such as mode used to work, time of departure, durations and others. In the last section, the respondent was asked to rate their perceptions on the sustainability attributes.

## 3. CHARACTERISTICS OF RESPONDENTS

Table 2 presents the characteristics of the respondents in terms of gender and education level. In general, there were more male motorcyclists compared to female which is consistent with the phenomenon in Malaysia. Most of the car users were with higher education qualification.

Table 2 Distribution of Respondents by Gender and Education Level

| | Gender | | | | Education Level | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Female | % | Male | % | Primary | % | Secondary | % | College/ University | % |
| Car User | 224 | 85.2 | 261 | 69.4 | 12 | 38.7 | 131 | 74.4 | 342 | 79.2 |
| Motorcyclist | 39 | 14.8 | 115 | 30.6 | 19 | 61.3 | 45 | 25.6 | 90 | 20.8 |
| Total | 263 | 100.0 | 376 | 100.0 | 31 | 100.0 | 176 | 100.0 | 432 | 100.0 |

Table 3 and Table 4 show the distribution of respondents by race and age group. Majority of the respondents were Malay making up 45% of total respondents. About 3% of the respondents were aged below 18 years old and above 55 years old, respectively.

Table 3 Distribution of Respondents by Race

| | Malay | % | Chinese | % | Indian | % | Others | % |
|---|---|---|---|---|---|---|---|---|
| Car User | 207 | 71.38 | 183 | 84.33 | 82 | 71.30 | 13 | 76.47 |
| Motorcyclist | 83 | 28.62 | 34 | 15.67 | 33 | 28.70 | 4 | 23.53 |
| Total | 290 | 100 | 217 | 100 | 115 | 100 | 17 | 100 |

Table 4 Distribution of Respondents by Age Group

| Age Group | Car User | | Motorcyclist | | Total | |
|---|---|---|---|---|---|---|
| | No. | % | No. | % | No. | % |
| < 18 yrs old | 11 | 61.11 | 7 | 38.89 | 18 | 100.0 |
| 18 – 24 yrs old | 166 | 67.21 | 81 | 32.79 | 247 | 100.0 |
| 25 – 30 yrs old | 146 | 78.07 | 41 | 21.93 | 187 | 100.0 |
| 30 – 44 yrs old | 110 | 87.30 | 16 | 12.70 | 126 | 100.0 |
| 45 -55 yrs old | 37 | 84.09 | 7 | 15.91 | 44 | 100.0 |
| > 55 yrs old | 15 | 88.24 | 2 | 11.76 | 17 | 100.0 |

## 4. RESULTS

From the previous findings (Ho et al., 2008), it was found that in general majority of the motorised users in Klang Valley were not favourable to the services offered by public transport. In choosing car or motorcycle, factors such as comfort and cleanliness, personal status and privacy, stress, safety, sending kids to school and irregular of working hours were identified playing significant role. On the other hand, realising the high accident rates involving motorcyclists and their pillion riders, many strategies were devised to tackle this problem. Thus, one of the series efforts in this research study was to investigate if the motorised users are willing to switch mode based on a list of incentives. The results showed that private vehicles users responded well to the incentives especially if the security systems were to be improved.

In this paper, eight sustainability attributes were identified to further examine motorised users' perceptions. The eight attributes were short-listed from the literature and pilot survey. Figure 1 shows the eight attributes which are classified under 2 categories namely sustainable transportation performance measures and traveller's perception.

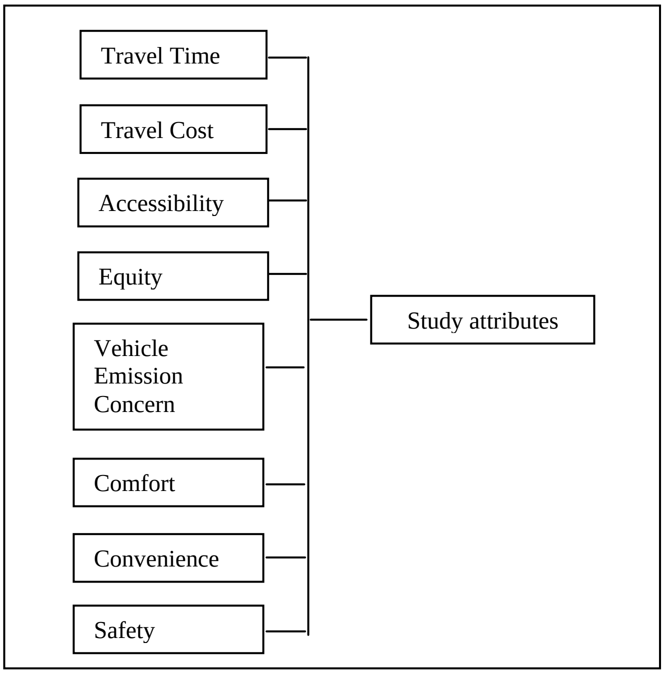



Figure 1 Attributes identified in this study

Table 5 presents the results of the mean score for the eight attributes as well as the t-test results. Respondents were asked to rate their travel time, expenses and other performance offered by their mode of transport on 10-point Likert scale as well as on 5 groups of linguistic variables (Very Low, Low, Moderate, High and Very High). It was found that all the attributes scored below 5 (average) of which total travel objectives to be able to achieved within 3 hours

scored the lowest (2.73 – 2.78). This finding is expected as with the dispersed development and serious traffic congestion in Klang valley, vehicle accessibility is relatively low compared to other well planning countries like Singapore and Tokyo. In terms of perceived security level offered by their mode of transport, it was found that motorcyclists viewed their mean of travel could offer better compared to car users. This may be true as in most of the office complexes, motorcycle parking lots are always near to entrance or management offices. This, in other words, provide higher security level to motorcyclists.

Table 5 Mean score for perceived attributes and t – test results by modes of transport

| Attribute | Mode | Mean | Standard Deviation | sig. |
|---|---|---|---|---|
| Perceived travel time spent to work | Car User | 3.02 | 0.79 | 0.104 |
| | Motorcyclist | 3.11 | 0.83 | |
| Perceived travel expenses to work | Car User | 3.19 | 0.73 | 0.099* |
| | Motorcyclist | 3.09 | 0.96 | |
| Total objectives achieved by travel mode | Car User | 2.73 | 0.94 | 0.652 |
| | Motorcyclist | 2.78 | 0.91 | |
| Perceived security level | Car User | 3.45 | 0.79 | 0.063* |
| | Motorcyclist | 3.52 | 0.95 | |
| Awareness on vehicle emission | Car User | 4.36 | 1.08 | 0.971 |
| | Motorcyclist | 4.18 | 1.17 | |
| Perceived comfort level | Car User | 3.36 | 0.84 | 0.065* |
| | Motorcyclist | 3.52 | 1.00 | |
| Perceived convenience level | Car User | 3.39 | 0.76 | 0.07* |
| | Motorcyclist | 3.63 | 0.89 | |

*Note: * significant at 0.1 level*

Figure 2 and Table 6 illustrate the perceived travel time used to travel to work for car users and motorcyclists. Approximately 63% of the car users in Klang Valley labelled that their daily commute time as moderate. As motorcycle is known as a highly manoeuvrable vehicle, more motorcyclists perceived their travel time higher compared to car users.

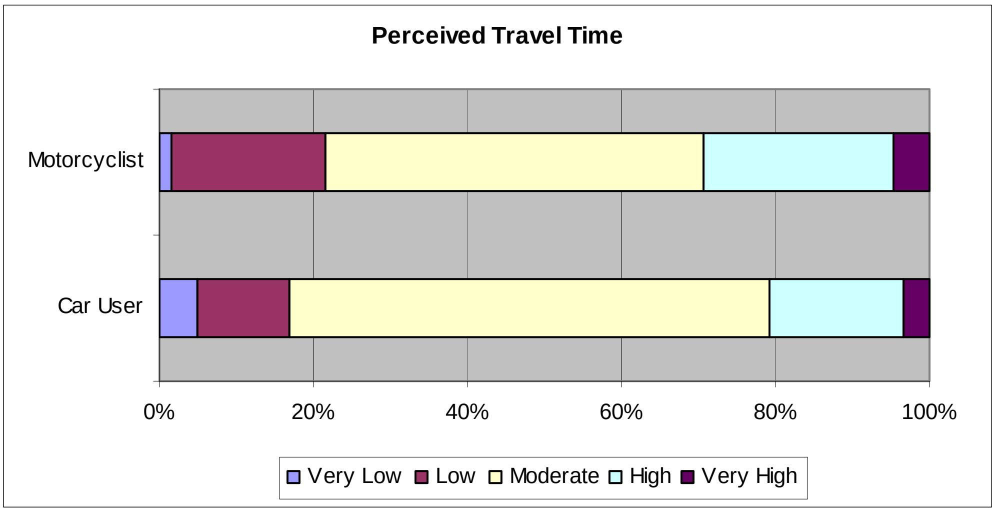



Figure 2 Perceived travel time by drivers and motorcyclists

Table 6 Perceived Travel time

| Perceived Travel Time | Very Low (%) | | Low (%) | | Moderate (%) | | High (%) | | Very High (%) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | M | C | M | C | M | C | M | C | M | C |
| <15 min | 0.0 | 2.2 | 1.5 | 6.5 | 6.2 | 19.6 | 0.0 | 0.0 | 0.0 | 0.0 |
| 15-30 min | 1.5 | 2.7 | 9.2 | 3.8 | 24.6 | 28.3 | 7.7 | 7.6 | 0.0 | 0.0 |
| 31-45 min | 0.0 | 0.0 | 7.7 | 1.1 | 9.2 | 10.9 | 7.7 | 3.8 | 1.5 | 0.0 |
| 46-60 min | 0.0 | 0.0 | 1.5 | 0.5 | 9.2 | 2.7 | 7.7 | 4.9 | 1.5 | 1.6 |
| >60 min | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.1 | 1.5 | 1.1 | 1.5 | 0.5 |

Note: M=motorcyclists; C = car users

Travel expenses were classified into 6 groups as shown in Table 7. Majority of the travellers rated that their monthly expenditure on transport as moderate (Figure 3). Near to 60% of the motorcyclists spent less than RM 300 per month and they rated their expenses between Very Low to Moderate. As travel with car would incur higher cost compared to travelling with motorcycle, generally car users' acceptance level on travel expenses were higher compared with their counterparts who were using motorcycles.

Table 7 Perceived Travel Expenses

| Travel Expenses | Very Low (%) | | Low (%) | | Moderate (%) | | High (%) | | Very High (%) | |
|---|---|---|---|---|---|---|---|---|---|---|
| | M | C | M | C | M | C | M | C | M | C |
| < RM100 | 1.5 | 1.1 | 7.7 | 1.1 | 3.1 | 5.5 | 0.0 | 0.0 | 0.0 | 0.0 |
| RM 101-200 | 3.1 | 1.1 | 6.2 | 2.2 | 23.1 | 11.5 | 0.0 | 3.8 | 0.0 | 0.0 |
| RM 201-300 | 0.0 | 0.0 | 3.1 | 2.7 | 10.8 | 16.4 | 3.1 | 4.4 | 4.6 | 0.0 |
| RM 301-400 | 0.0 | 0.0 | 0.0 | 0.5 | 10.8 | 11.5 | 7.7 | 7.7 | 3.1 | 0.5 |
| RM 401-500 | 0.0 | 0.0 | 0.0 | 0.5 | 3.1 | 8.7 | 3.1 | 4.4 | 1.5 | 1.1 |
| > RM500 | 0.0 | 0.0 | 0.0 | 0.0 | 3.1 | 8.7 | 0.0 | 3.8 | 1.5 | 2.2 |

Note: M=motorcyclists; C = car users

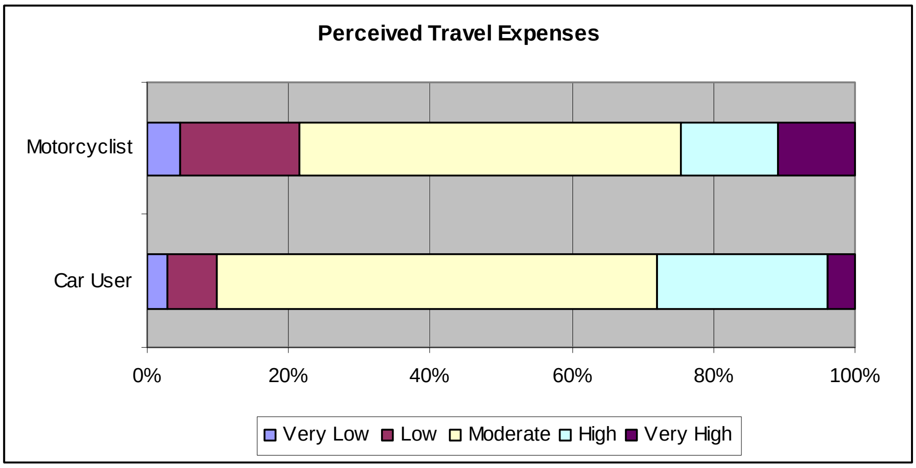



Figure 3 Perceived Travel Expenses

Travel objective in this study is defined as number of destination or objectives that can be achieved within 3 hours. Table 7 and Figure 4 present the travellers' perceptions on travel objectives achieved within 3 hours in Klang Valley. Given the small size and high accessibility, motorcyclists were able to achieve more travel objectives compared to car drivers.

Table 7 Travel Objectives achieved

| Number of destinations | Very Low (%) | | Low | | Moderate | | High | | Very High | |
|---|---|---|---|---|---|---|---|---|---|---|
| | M | C | M | C | M | C | M | C | M | C |
| 1-2 | 7.7 | 7.1 | 3.1 | 4.9 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 3-4 | 0.0 | 0.5 | 24.6 | 27.2 | 10.8 | 2.7 | 0.0 | 1.6 | 0.0 | 0.0 |
| 5-6 | 0.0 | 0.0 | 0.0 | 1.1 | 32.3 | 39.1 | 7.7 | 11.4 | 0.0 | 1.6 |
| 7-8 | 0.0 | 0.0 | 0.0 | 0.0 | 3.1 | 0.5 | 7.7 | 0.0 | 0.0 | 2.2 |
| 9-10 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 6.2 | 0.0 |

Note: M=motorcyclists; C = car users

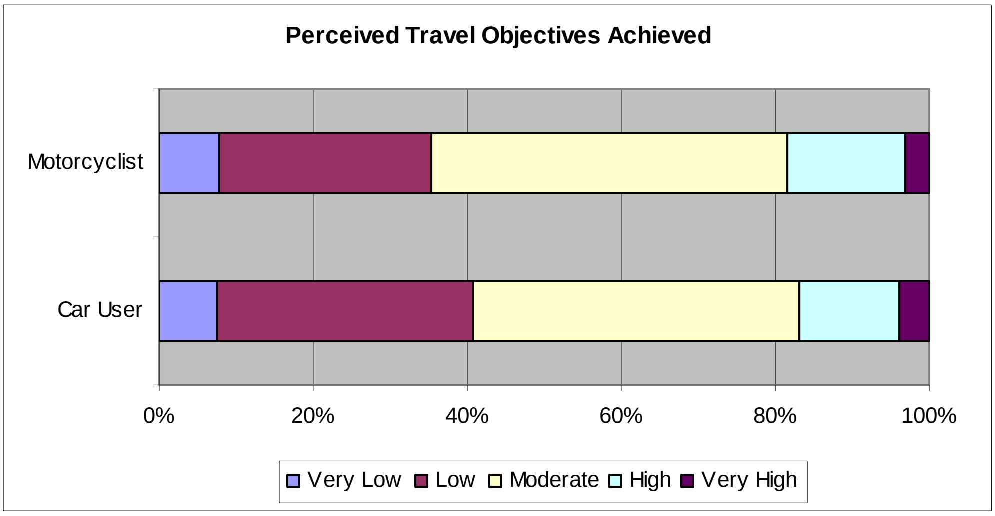



Figure 4 Travel objectives achieved

Perceived security level in this case refers to the availability of surveillance equipment, security officers and others to provide safe environment for travellers. About 20% of the motorcyclists felt that travelling with motorcycles was very much secured while only 7% of the car users shared the same opinion.

Table 8 Perceived Security Level

| Scale | Very Low (%) | | Low | | Moderate | | High | | Very High | |
|---|---|---|---|---|---|---|---|---|---|---|
| | M | C | M | C | M | C | M | C | M | C |
| 1-2 | 1.5 | 1.6 | 3.1 | 2.2 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 3-4 | 1.5 | 0.0 | 3.1 | 4.9 | 10.8 | 1.6 | 0.0 | 0.0 | 0.0 | 0.0 |
| 5-6 | 0.0 | 0.0 | 0.0 | 0.5 | 30.8 | 38.6 | 1.5 | 1.6 | 0.0 | 0.0 |
| 7-8 | 0.0 | 0.0 | 0.0 | 0.0 | 1.5 | 1.6 | 29.2 | 39.1 | 0.0 | 0.0 |
| 9-10 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.6 | 16.9 | 6.5 |

Note: M=motorcyclists; C = car users

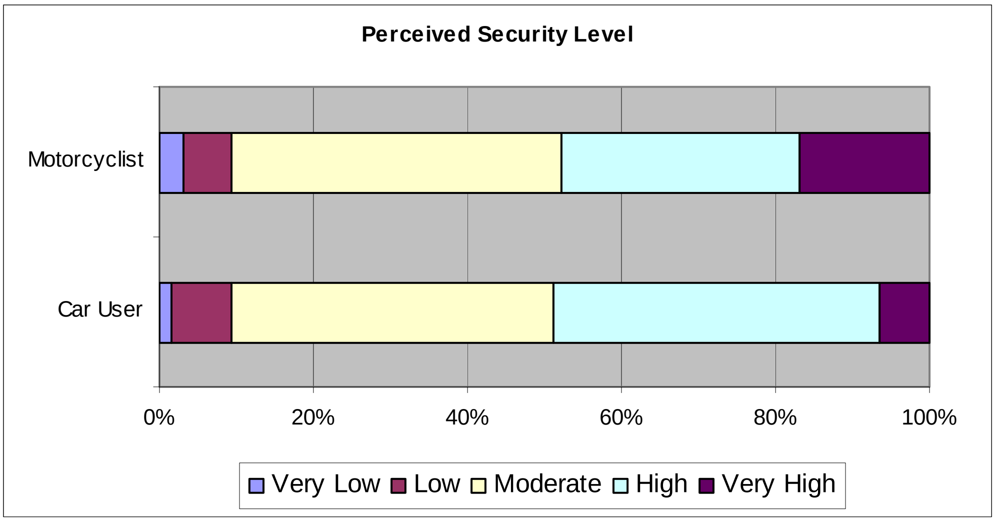



Figure 5 Perceived security level

Figure 6 and 7 illustrate the perceived comfort and convenience level offered by their mode of transport, respectively. Generally, motorcyclists viewed that their mode of transport was more convenient while car was rated more comfortable compared to motorcycles (see also Tables 9 and 10).

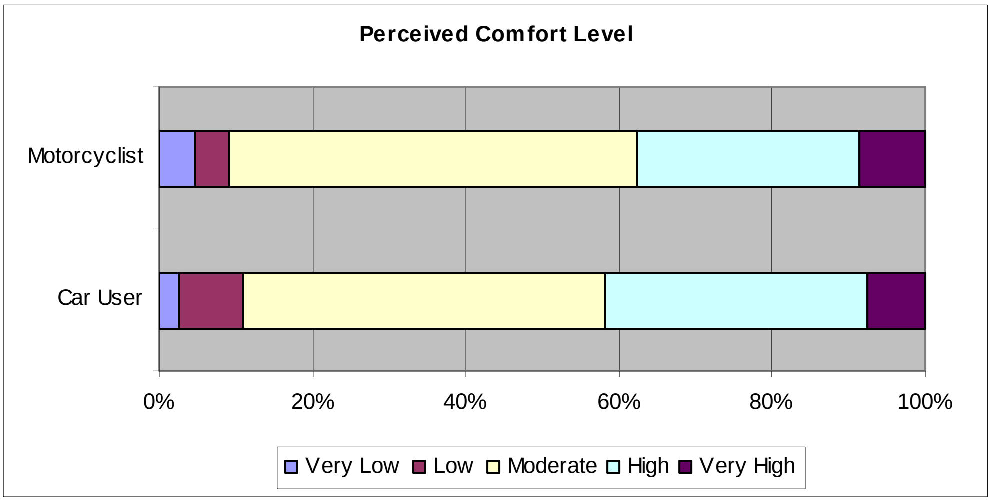



Figure 6 Perceived comfort level

Table 9 Perceived comfort level

| Scale | Very Low (%) | | Low | | Moderate | | High | | Very High | |
|---|---|---|---|---|---|---|---|---|---|---|
| | M | C | M | C | M | C | M | C | M | C |
| 1-2 | 8.6 | 2.7 | 0.0 | 2.2 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 3-4 | 0.0 | 0.0 | 6.6 | 5.4 | 11.8 | 2.2 | 0.0 | 0.0 | 0.0 | 0.0 |
| 5-6 | 0.0 | 0.0 | 1.5 | 0.5 | 29.2 | 42.4 | 1.5 | 0.0 | 0.0 | 0.0 |
| 7-8 | 0.0 | 0.0 | 0.0 | 0.0 | 16.2 | 2.7 | 16.2 | 27.2 | 0.0 | 0.0 |
| 9-10 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.5 | 6.5 | 6.9 | 7.6 |

Note: M=motorcyclists; C = car users

Table 10 Perceived convenience Level

| Scale | Very Low (%) | | Low | | Moderate | | High | | Very High | |
|---|---|---|---|---|---|---|---|---|---|---|
| | M | C | M | C | M | C | M | C | M | C |
| 1-2 | 1.5 | 1.6 | 1.5 | 2.7 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 3-4 | 0.0 | 0.0 | 4.6 | 2.7 | 4.6 | 2.7 | 0.0 | 0.0 | 0.0 | 0.0 |
| 5-6 | 0.0 | 0.0 | 0.0 | 0.0 | 29.2 | 46.7 | 1.5 | 0.0 | 0.0 | 0.0 |
| 7-8 | 0.0 | 0.0 | 0.0 | 0.0 | 3.1 | 2.2 | 29.2 | 23.9 | 0.0 | 0.0 |
| 9-10 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 7.7 | 10.9 | 16.9 | 6.5 |

Note: M=motorcyclists; C = car users

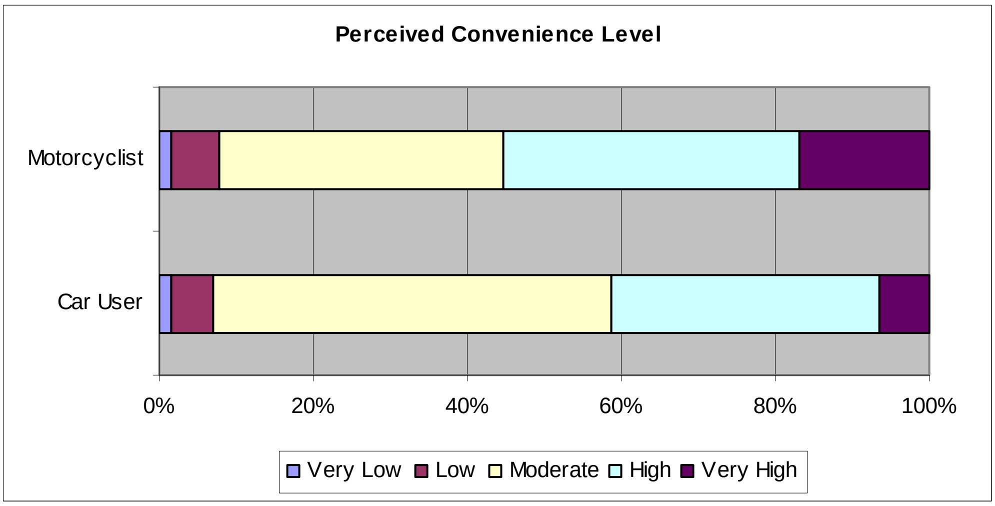



Figure 7 Perceived convenience level

Figure 8 shows the rating of travellers' awareness on air pollution due to vehicular emission. Though about 50% of the car users realised that vehicular emission had a very high effect on the air pollution, however, it did not deter or reduce their intention to travel with car. This phenomenon is not sustainable and warrant further investigation to explore their travel habits with private vehicles.

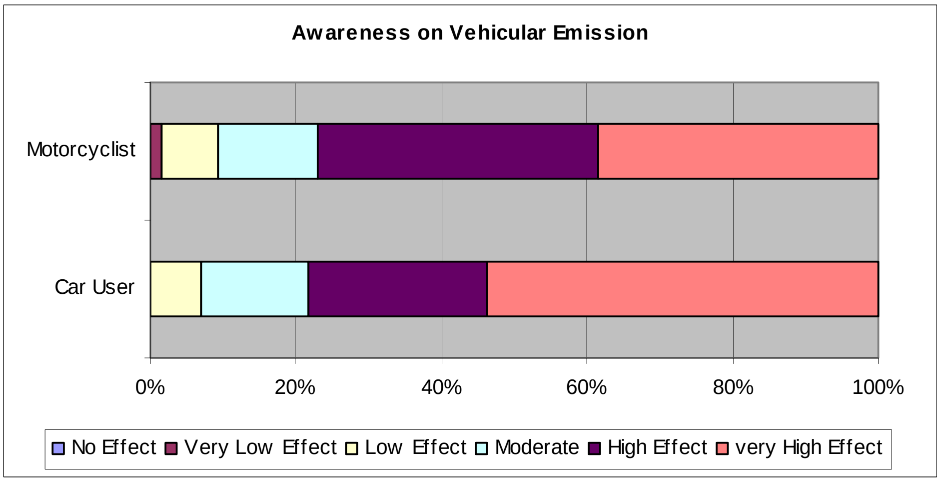



Figure 8 Awareness on vehicular emission

Logistic regression model was developed to provide an insight into the factors influencing the motorcycle use. The dependent variable is dichotomous: car use or motorcycles use. The study variables include gender, age, education level, occupation, income level, number of family members that reside together, time to reach destination distance to destination and travel expenses. Description of variables used in logistic model was shown in Table 11.

Table 11 Description of variables

| Variable | Level |
|---|---|
| Gender | Male = 1; Female= 0, |
| Age | Continuous variable |
| Education | Tertiary=1; otherwise = 0 |
| Occupation | Professional=1; otherwise=o |
| Income level | Continuous variable |
| Number of family member | Continuous variable |
| Time to reach destination | Continuous variable |
| Distance to distance | Continuous variable |
| Travel Expenses | Continuous variable |

Table 12 Logistic Regression analysis

| Variable | B | S.E. | Wald | Sig. | Odd Ratio (OR) |
|---|---|---|---|---|---|
| Gender | -.944 | .321 | 8.637 | .003* | 2.570 |
| Age | .008 | .017 | .216 | .642 | 1.008 |
| Education | -.103 | .319 | .104 | .747 | 1.108 |
| Occupation | 1.108 | .777 | 2.036 | .154 | 0.330 |
| Income level | -.001 | .000 | 14.335 | .000* | 0.999 |
| Number of family member | .053 | .081 | .427 | .514 | 1.055 |
| Time to reach destination | -.001 | .013 | .008 | .927 | 0.999 |
| Distance to workplace | -.040 | .015 | 6.843 | .009* | 0.961 |
| Travel Expenses | .000 | .000 | 1.533 | .216 | 1.000 |
| Constant | -.723 | 1.047 | .476 | .490 | 0.516 |
| Summary of Statistics | | | | | |
| Chi-square, df= | 66.78 (p<0.001) | | | | |
| Hosmer – Lemeshow test, $\chi^2$ | 0.426(df=8, p<0.01) | | | | |
| -2 log likelihood | 291.562 | | | | |
| Nagelkerke $R^2$ | 0.256 | | | | |
| Correctly classified car | 93.5% | | | | |
| Correctly classified motorcycle | 26.3 | | | | |
| Overall percentage correctly classified | 77.3% | | | | |
| * significant at 0.05 level | | | | | |

Table 12 presents the logistic regression analysis results for prediction of motorcycle use. The Hosmer and Lemeshow Goodness-of-fit Test result shows 0.426 indicating the model explained the data reasonable well. The Nagelkerke $R^2$ was in this model is 0.256. The results also indicate that the overall percent correctly classified were 77.3% predicted correctly. The variables such as gender, income level and distance to destination were found significant at the 0.05 level. Male travellers were found 2.6 times more likely to use motorcycle compared to female travellers. The further the work place or the longer commuting time the less likely travellers chose to use motorcycles which was shown by the negative coefficient signs.

Figures 9 and 10 show the motorcycle use probabilities against income level and distance to work place, respectively. Those who earn RM 3000 per month have 2.35% likelihood to use motorcycles and the likelihood reduced to 0.5% for those who earn RM 4500 per month.

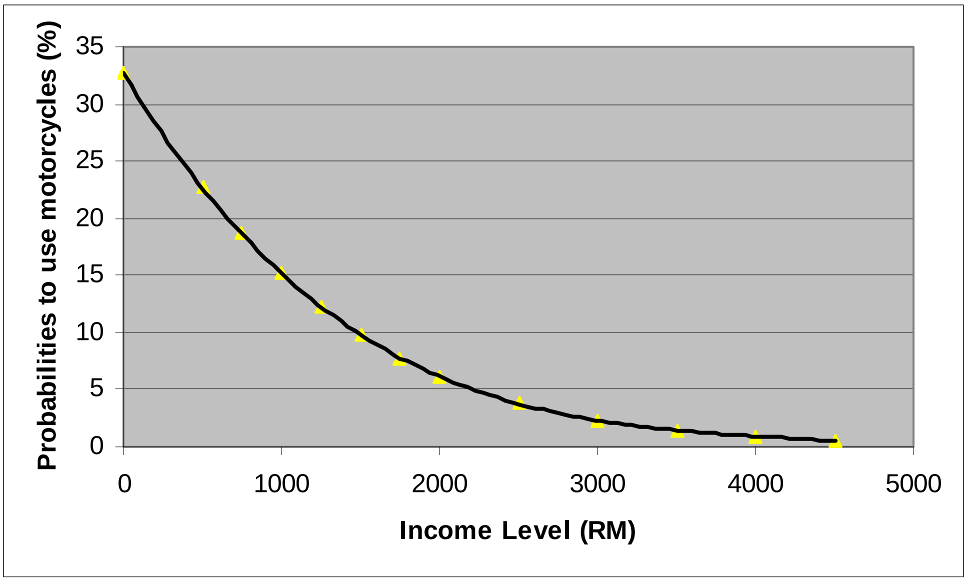


Figure 9 Effect of income level on probabilities to use motorcycles

The longer the travel working distance, the lesser likelihood would be a person to use motorcycle. The probabilities to use motorcycle for those who work as far as 50 km from his house are 6.2%.

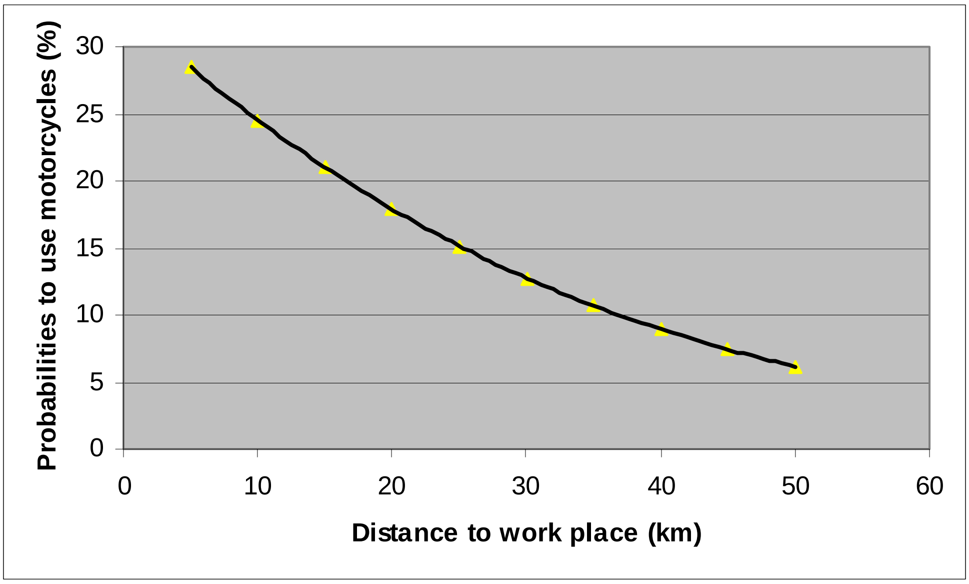


Figure 10 Effect of distance to work place on probabilities to use motorcycles

## 5. Conclusion

This study has identified that gender, income level and distance to work place were significant in motorcycle use. Based on the model, male travellers were 2.6 times more likely to use motorcycles compared to female travellers. Though the likelihood to use motorcycles decrease when the distance to work increase, it is expected that motorcycles will continue to be one of the major modes of transportation in city centres during the world economic downturn. Petrol price hike is also seen to be one of the stimulants for traveller to use motorcycles as this phenomenon was observed when petrol price in Malaysia increased by 41% in June 2007. Private vehicle users expressed interests to shift to public transports if given incentives. The findings from this study though is not served as planning guide, but can provide a better perspective to the future planning and development of traffic system.

**Refences**

Ho, J.S., Leong, L.V. and Mohd Sadullah, A.F., 2008. Understanding travel behabiour: An important approach to switch private car users to public transport. Malaysian Universities Transport Research Forum Conference 2008, UTM.

Johari, N.M., Mohd Sirat, S. R., Subramaniam, S. N. Yellappan, K., Suret, S. and Radin Umar, R.S. 2007. Measuring the effectiveness of the community based safety helmet programme in Kuang. Malaysian Institute of Road Safety Research, MRR5.

Leong, L.V. and Mohd Sadullah, A.F., 2007. A study on the motorcycle ownership: A case study in Penang State, Malaysia. Proceedings of the Eastern Asia Society for Transportation Studies, Vol. 6.

Mohamad, J., 2003. Rail transit development and urban regeneration in Kuala Lumpur. Planning Congress Adelaide.

Road Transport Department, Transport Statistics. [Online: 26 July 2007] http://www.mot.gov.my/BM/stat/darat.htm

The Star, 2006. Putting the best route forward. Article on press dated 24 September 2006.

# PREFERENCE OF TRAVELLERS FOR SUSTAINABLE TRANSPORTATION PLANNING OBJECTIVES IN KLANG VALLEY, MALAYSIA

**Lee Vien LEONG, Jen Sim HO and Ahmad Farhan MOHD SADULLAH**
*School of Civil Engineering, Universiti Sains Malaysia, Engineering Campus,*
*14300 Nibong Tebal, Seberang Perai Selatan,*
*Pulau Pinang, Malaysia*
*celeong@eng.usm.my*

**ABSTRACT**

Achieving sustainable transportation is a global aim today. However, the actual implementation of sustainable principles is severely challenging the decision-makers. Various efforts attempted to reduce the ownership of private vehicles have not been fruitful. The failures may be attributed to the fact that public transport services are unable to provide effective travel needs as offered by cars. This paper uses stated preference method to examine the travellers' preferences for a set of hypothetical sustainable transportation strategies over different transport options. Factors evaluated in this study cover three basic sustainability dimension namely social, economic and environmental factors. A disaggregate choice model based on the data collected was developed. This model would provide better understanding of travel demands and their constraints in Klang Valley, Malaysia. In addition, analysis to determine the value of time was also conducted and these findings have been found to be useful for future planning and development of sustainability measures.

## 1. INTRODUCTION

Since the Brudtland Report was tabled in 1987 (WCDE, 1987), the concept of sustainability which was further culminated during the Earth Summit (1992) held in Rio de Janeiro has been increasingly gaining attention among the policy makers and researchers (Andriantiatsaholiniaina et al., 2004). Many definitions on different subject have been developed (Barbier, 1987, WCDE, 1987, Common and Perrings, 1992, Pearce et al., 1989). Among them, one as defined in the Brudtland Report is "Development that meets the needs of the present without compromising the ability of future generations to meet their own needs" (WCDE, 1987).

Transportation plays an important role in sustainable development, since transport activities tend to be highly resource intensive and have numerous external costs. For example, the transportation sector consumes more than 60% of all petroleum products globally, despite efforts by many governments to encourage the substitution of other fuels (NRTEE, 1997). The concept of sustainable transport system covers varies of issues. Banister (2000) in his report addressed seven key issues as a result of transportation activities. They are as follows:

(i) Congestion - An average, speeds in some cities have been decreasing by 5% per decade (EFTE, 1994).
(ii) Increasing air pollution - According to the standard recommended by WHO (1997), air quality in many cities have been exceeded.
(iii) Traffic noise – It is estimated that about 15% of the population in developing countries are exposed to high levels of noise (OECD, 1995).
(iv) Road safety – Around the world, there are 250, 000 deaths and 10 million of injuries resulted from accidents (Downey, 1995).
(v) Degradation of urban landscapes.
(vi) Use of space by traffic.
(vii) Global warming – Transport accounts for 25% of $CO_2$ emissions.

There are many definitions on sustainable transportation. A sustainable transport system as defined by The Centre for Sustainable Transportation (Gilbert and Tanguay, 2000) is as follows:

- Allows the basic access and development needs of individuals, companies and societies to be met safely and in a manner consistent with human and ecosystem health, and promotes equity within and between successive generations.
- Is affordable, operates fairly and efficiently, offers choice of transport mode, and supports a competitive economy, as well as balanced regional development.
- Limits emissions and waste within the planet's ability to absorb them, uses renewable resources at or below their rates of generation, and uses non-renewable resources at or below the rates of development of renewable substitutes while minimizing the impact on the use of land and the generation of noise.

The progress of sustainable transportation is in fact very unsatisfactory (Ziestman et al., 2003). The concept has been regarded as one of the most arguable but least implemented concept in urban and transportation planning (Lindquist, 1998). Among the problems identified are inadequate understanding and recognition of increasingly important social, economic, environmental and public policy issues, short of quantified measures that can be used for monitoring and decision making as well as lacking of co-ordination between decision-makers, planners and other stakeholders (Ziestman and Rilett, 2000).

The key variables embrace in sustainable transportation concept are accessibility for all, social equity (the poor and the disadvantaged) and ecological sustainability. Promoting public transportation is a way to reduce the use of private cars in urban and hereby the adverse impacts of transportation (Banister, 2000). Increase of transit use is definitely associated with overall sustainability. A study conducted by Sinha (2003) shows that a 10% increase in transit boardings per capita per year could decrease transportation energy consumption per capita per year by about 1,700 million joules or a 10% increase in the transit share of work trips can decrease $CO_2$ emissions per capita per year by about 130 kg. To achieve social equity, Osula (1998) notes that through government subsidization, the travel expenditure burden of the poor and disadvantage can be eased. Nevertheless, it is seen that the statistic of car ownership is still increasing in every part of the world and the trend is moving away from sustainability.

The CfIT (2001) which is one of the relatively few studies to examine the attitudes of public transport users, pedestrians and cyclists as well as car users noted that half of the population would reduce travelling by car if the local bus services were better, a third if local rail services were better, and a quarter if local conditions for walking were better. Surveys have also proposed that a general dissatisfaction with the price, safety and reliability of public transport and a tendency to exaggerate problems with staff attitudes, frequency, availability of seats, cleanliness of vehicles, fast, comfortable, convenient and affordable services (Bonsall et al., 2005).

## 2. PROBLEM STATEMENT

Klang Valley Region spans across approximately 2,843 square kilometres. The entire region of Klang Valley constitutes of Gombak District, Hulu Langat District, Federal Territory of Kuala Lumpur, Klang District and Petaling District. The total population in Klang Valley has exceeded 6 millions in year 2006. Being the epic-center of economic growth for the country, Klang Valley is the fastest growing and vibrant region.

The public transport services in Klang Valley are served by rails, buses and taxis. Two Light Rail Transits (STAR and PUTRA LRT), KTM Komuter service and monorail system provide intra-city travel facilities. The bus service main provider, Rapid KL currently operates 161 bus routes with 908 buses in operation. Together with more than 13000 taxis in Klang Valley, the relevant authorities strive to provide quick, affordable and comfortable travel options. Despite the improvements to the public transport services, statistics show that from 1985 to 1997, the percentage of public transport modal share has declined from 34.3% to 19.7% (CHKL, 2003). These low public transport riderships are mainly associated with the inconvenient, inefficient, unreliable and uncomfortable services (Kiggundu, 2005). According to a recent study initiated by Syarikat Prasarana Nasional Bhd (SPNB) (The Star, 2006), there were 2.2 million private vehicles moving into the city daily. The excessive influx of private vehicles into Klang Valley has also brought the adverse impacts such as traffic congestion, air pollution that creates considerable pressure on the road network systems. It was estimated that 80 percent of the pollutants came from motor vehicle sources (Yahya and Sadullah, 2002). The traffic congestion condition in city centre has continued to deteriorate. A study by

Barter (1999) found that average public transport speeds in the Klang Valley are only 16km/hr compared to 26 km/hr in Singapore and 28km/hr in Hong Kong (Barter, 1999). In terms of traffic fatalities, it was recorded that there were about 4.3 accident fatalities case for every 10, 000 registered vehicles (Marjan et al., 2007).

Concerns over the low public transport riderships and sustainability of transportation in Klang Valley have caught the attention of various decision makers, researchers and planners. Countermeasures include new LRT extensions to congested areas, fare ticket promotion, increase service frequencies, exclusive bus lanes and others. However, the demand on the public transport is still not encouraging.

## 3. STUDY VARIABLES

Any sustainable transportation policy would not success if the policy makers do not understand traveller's demand and choice. The task to identify the most influencing factors is thus a challenging task. Based on thorough review of literature and pilot survey, eight variables were selected to represent the sustainable transportation system which envelops three major dimensions: economic, environmental and social. Figure 1 shows the framework and study variables of this study.

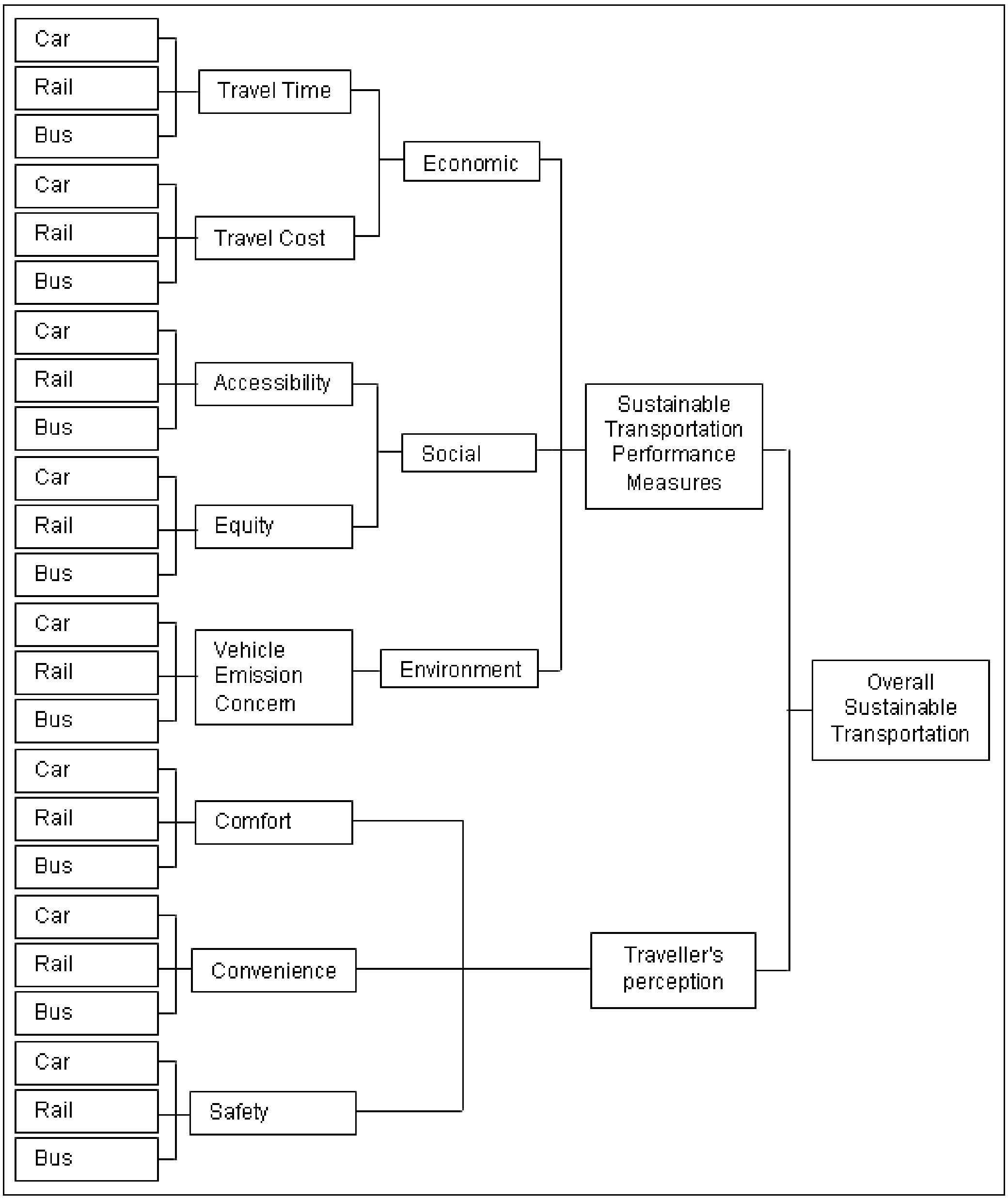



Figure 1: Study Framework

## 4. SURVEY DESIGN AND METHODOLOGY

To explore the effect of different policies on traveller's choice, stated preference (SP) approach was employed. SP experiments show a set of choices in relative to other modes of transport. Table 1 presents policy tools designed for this study. Each policy tool has three levels. Headway, walking distance to public transport service, comfort and convenience are not applicable to car users. The reason comfort and convenience level not tested on car users is that car is often viewed as more comfortable and convenience as compared to public transport. Travel objectives in this case refer to the number of destinations that are able to be reached within 3 hours while environmental awareness addresses the present air pollution level due to vehicular.

Table 1: Attributes and levels used for the mode choice showcards

| Attribute | Mode | | |
|---|---|---|---|
| | Car | Bus | Rail |
| IVT | 30, 45, 75 min | 50, 60, 75 min | 15, 20, 30 min |
| Headway | - | 5, 15, 30 min | 4, 10, 15 min |
| Walking distance to public transport service | - | 100m, 250m, 450m | 100m, 250m, 450m |
| Petrol | 1.60, 1.80, 1.90 per liter | - | - |
| Parking | RM 3, 5, 12 per entry | - | - |
| Toll | RM 0, 4, 5 per day | - | - |
| Travel objectives/ accessibilty | 3, 4, 5 | 3, 4, 5 | 3, 4, 5 |
| Environment awareness | Low, Medium, High | Low, Medium, High | Low, Medium, High |
| Security level | Low, Medium, High | Low, Medium, High | Low, Medium, High |
| Comfort level | - | Low, Medium, High | Low, Medium, High |
| Convenience level | - | Low, Medium, High | Low, Medium, High |

In this SP survey study, eight selected sustainable transportation attributes were tested over three modes of transportation which are car, bus and rail. This factorial design for all the attributes: $7^3$ (car) x $10^3$ (bus) x $9^3$ (rail) has resulted in 2.5 x $10^8$ combinations. It is impossible to carry all the combinations in survey. Thus, confounding factorial design was adopted. Confounding factorial design is a technique for arranging a complete factorial experiment in blocks, in which the block size is smaller than the number of treatment combinations in one replicate (Montgomery, 1997).

Table 2 summaries the fractional factorial design. For illustration purpose, consider car mode with 7 attributes at 3 levels ($3^7$) and is represented by 0 (low), 1 (intermediate) and 2 (high). To reduce the number of combinations, $3^{7-3}$ or also known as $\frac{1}{3^3}$ fraction is chosen. This fraction needs 4 independent generators and the remaining 3 generators are 3-factors interactions, for instance E=ABC. To complete all the design, the basic design for the 4 independent generators $3^4$ was first developed by using MINITAB software. Generators E, F and G were derived from the summation of three-factor interaction. A total of 27 scenarios were created based on the general rule of thumb of confounding (Montgomery, 1997) that is to confound the highest-order of interaction with blocks. All the main attribute effects were independent and orthogonal.

Table 2: Fractional Factorial Design

| Mode of transport | Factorial design | Fractional Factorial Design | Factorial Effect |
|---|---|---|---|
| Car | $3^{7}$ | $3^{7-3}$ | A, B, C, D, E=ABC, F=BCD, G=ACD |
| Bus | $3^{10}$ | $3^{10-6}$ | A, B, C, D, E=ABC, F=BCD, G=ACD, H=ABD, I=ABCD, J=AB |
| Rail | $3^{9}$ | $3^{9-5}$ | A, B, C, D, E=ABC, F=BCD, G=ACD, H=ABD, I=ABCD |

Figure 2 presents an example showcard of an SP choice scenario. Each respondent was presented with 2 different scenarios and was asked "if these travel options were available, which mode would you choose?".

| 27 | A : Travel by Car | B : Travel by Bus | C : Travel by Rail |
|---|---|---|---|
| | In-Vehicle-time: **30 min** | Vehicle run every: **5 min**<br>In-Vehicle-Time: **50 min** | Vehicle run every: **15 min**<br>In-Vehicle-Time: **30 min** |
| | Petrol Price: **1.80 per litre**<br>Toll: **RM 5 per day**<br>Parking: **RM 12 per entry** | Fare: **RM 1.80 per trip** | Fare: **RM 1.70 per trip** |
| | Destination/objectives: **3 places** | Destination/objectives: **3 places** | Destination/objectives: **2 places** |
| | Security Level: **Medium** | Security Level: **Medium** | Security Level: **Medium** |
| | | Walking Distance: **100 m** | Walking Distance: **250 m** |
| | Pollution due to vehicles : **High** | Pollution due to vehicles : **Medium** | Pollution due to vehicles : **Medium** |
| | | Comfort Level: **Low** | Comfort Level: **Medium** |
| | | Convenience Level: **Medium** | Convenience Level: **Low** |

If theses choices were available, which would you choose?

Travel by Car ☐

Travel by Bus ☐

Travel by Rail ☐

Figure 2: Example of showcard

Besides the mode choice attributes, social-demographic characteristics of each respondent was also examined revealed preference (RP) survey. These characteristics include gender, age, income, occupation, level of education, vehicle ownership, monthly travel expenses and travel duration. The characteristics were described in more detail in Table 3.

Table 3: Mode choice case study variable labels

| Variable Name/ Description | Labels |
|---|---|
| ID | Respondent ID |
| Gender (RP) | 1 = Female<br>2 = Male |
| Race (RP) | 1 = Malay<br>2= Chinese<br>3 = Indian<br>4 = Others |
| Age group (RP) | 1 = 18 – 24 years<br>2 = 25 – 30 years<br>3 = 31 – 44 years<br>4 = 45 – 55 years<br>5 = > 56 years |
| Education level (RP) | 1 = Primary<br>2 = Secondary<br>3 = College / tertiary |
| Occupation (RP) | 1 = Student<br>2 = Technician<br>3 = Executive / administration<br>4 = Managerial / Professional<br>5 = Self – employed<br>6 = Retiree<br>7 = Others |
| Monthly income (RP) | 1 = < RM 1000<br>2 = RM 1001- 1500<br>3 = RM 1501 – RM 2000<br>4 = RM 2001 – RM 3000<br>5 = RM 3001 – RM 4000<br>6 = RM 4001 – RM 5000<br>7 = RM > 5000 |
| Total expenses (RP) | As per specified by respondent |
| In-vehicle time (SP) | As per Table 1 |
| Headway(SP) | As per Table 1 |
| Walking distance to public transport service (SP) | As per Table 1 |
| Fare (SP) | As per Table 1 |
| Cost for petrol (SP) | As per Table 1 |
| Parking (SP) | As per Table 1 |
| Toll (SP) | As per Table 1 |
| Travel objectives/ accessibility (SP) | As per Table 1 |
| Environment awareness (SP) | As per Table 1 |
| Security level (SP) | As per Table 1 |
| Comfort level (SP) | As per Table 1 |
| Convenience level (SP) | As per Table 1 |

## 5. DATA CHARACTERISTICS

A total of 635 survey forms were collected but after screening, 44 incomplete forms were discarded. Table 4 and Table 5 show the distribution of respondents by gender and race respectively. Male respondents comprised of 50.6% and 43% of the total respondents were Malays. Chinese respondents composed of 34% followed by Indians, 18.3% and others, 4.7%

Table 4: Distribution of respondents by gender

| Gender | Frequency | Percentage (%) |
|---|---|---|
| Female | 292 | 49.4 |
| Male | 299 | 50.6 |
| Total | 591 | 100.0 |

Table 5: Distribution of respondents by race

| Gender | Frequency | Percentage (%) |
|---|---|---|
| Malay | 254 | 43.0 |
| Chinese | 201 | 34.0 |
| Indian | 108 | 18.3 |
| Others | 28 | 4.7 |
| Total | 591 | 100.0 |

Table 6 shows the distribution of respondents by age group. Based on Figure 6, most of the respondents (33.5%) aged between 31- 41 year old.

Table 6: Breakdown of respondents by age group

| Age Group | Frequency | Percentage (%) |
|---|---|---|
| 18 – 24 years | 153 | 25.9 |
| 25 – 30 years | 176 | 29.8 |
| 31 – 41 years | 198 | 33.5 |
| 45 – 55 years | 56 | 9.5 |
| > 55 years | 8 | 1.4 |
| Total | 591 | 100.0 |

Table 7 shows the distribution of respondents' income level. Based on Table 7, about one – quarter of the respondents earned between RM 2001 – RM 3000 per month.

Table 7: Respondents' income level

| Income | Frequency | Percentage (%) |
|---|---|---|
| < RM 1000 | 86 | 14.6 |
| RM 1001 – RM 1500 | 101 | 17.1 |
| RM 1501 – RM 2000 | 96 | 16.2 |
| RM 2001 – RM 3000 | 145 | 24.5 |
| RM 3001 – RM 4000 | 77 | 13.0 |
| RM 4001 – RM 5000 | 39 | 6.6 |
| > RM 5001 | 47 | 8.0 |
| Total | 591 | 100.0 |

## 6. MODE CHOICE ANALYSIS

In this study, Multinomial Logit (MNL) model was used to represent the decision making procedure of an individual, in making a choice from a set of alternatives based on various influencing factors. The LIMDEP NLOGIT software was used to develop this model. As each respondent was presented 2 choice sets, there were a total of 1182 cases.

IN MNL, utility function for an alternative represents a linear equation corresponding to the functional relationship between various attributes with that particular alternative (Hensher et al., 2005). The basic form of utility functions for car, bus and rail used in the mode choice analysis are as follows:

$$U(car) = constant + \beta_1 car * variable\text{-}1 + \beta_2 car * variable\text{-}2 + \ldots\beta_n car * variable\text{-}n \quad (1)$$
$$U(bus) = constant + \beta_1 bus * variable\text{-}1 + \beta_2 bus * variable\text{-}2 + \ldots\beta_n bus * variable\text{-}n \quad (2)$$
$$U(rail) = \beta_1 rail * variable\text{-}1 + \beta_2 rail * variable\text{-}2 + \ldots\beta_n rail * variable\text{-}n \quad (3)$$

Based on the attributes listed in Table 3, the preliminary assumptions of the utility functions for the three modes are shown in equation (4), (5) and (6). Attributes related to socio-demographic characteristics such as gender, race, age, education level, occupation and income will be invariant across alternatives. This invariance means that the parameter for that particular variable cannot be estimated for each and every utility function within the model. Therefore, in order to establish some variance, parameters related to socio-demographic characteristics can only be estimated for n–1 alternatives where n is the number of alternatives in the model (Hensher, 2005).

$$U(car) = a_1 + a_2{*}GENDER + a_3{}^*RACE + a_4{}^*AGE + a_5{}^*EDU + a_6{}^*OCCU - a_7{}^*INCOME - a_8{}^*CST - a_9{}^*PARK - a_{10}{}^*TOLL - a_{11}{}^*IVT_{car} - a_{12}{}^*EXPRP + a_{13}{}^*OBJ_{car} + a_{14}{}^*ENVIRON_{car} + a_{15}{}^*SECURITY_{car} + a_{16}{}^*COMFORT_{car} + a_{17}{}^*CONV_{car} \quad (4)$$

$$U(bus) = a_{18} + a_2 GENDER + a_3{}^*RACE + a_4{}^*AGE + a_5{}^*EDU + a_6{}^*OCCU - a_7{}^*INCOME - a_8{}^*FARE_{bus} - a_{11}{}^*IVT_{bus} + a_{13}{}^*OBJ_{bus} + a_{14}{}^*ENVIRON_{bus} + a_{15}{}^*SECURITY_{bus} + a_{16}{}^*COMFORT_{bus} + a_{17}{}^*CONV_{bus} - a_{19}{}^*HEADWAY_{bus} - a_{20}{}^*WALK_{bus} \quad (5)$$

$$U(rail) = -a_8{*}FARE_{rail} - a_{11}{}^*IVT{*}_{rail} + a_{13}{}^*OBJ_{rail} + a_{14}{}^*ENVIRON_{rail} + a_{15}{}^*SECURITY_{rail} + a_{16}{}^*COMFORT_{rail} + a_{17}{}^*CONV_{rail} - a_{19}{}^*HEADWAY_{rail} - a_{20}{}^*WALK_{rail} \quad (6)$$

where

| | | |
|---|---|---|
| GENDER | = | Gender |
| RACE | = | Race |
| AGE | = | Age group |
| EDU | = | Educational level |
| OCCU | = | Occupation |
| INCOME | = | Monthly income |
| CST | = | Cost for petrol |
| FARE | = | Fare |
| PARK | = | Parking |
| TOLL | = | Toll |
| EXPRP | = | Total expenses |
| OBJ | = | Travel objectives/ accessibility |
| ENVIRON | = | Environment awareness |
| SECURITY | = | Security level |
| COMFORT | = | Comfort level |
| CONV | = | Convenience level |
| HEADWAY | = | Headway |
| WALK | = | Walking distance to public transport service |

Several models were run and the results of the best fitted model were presented in Table 8. Attributes related to socio-demographic characteristics such as gender, race, age, education level and occupation was found to be statistically insignificant except income level which was found to be significant in explaining the mode choice behaviour. This is consistent with the findings by various researchers such as Pendyala et al. (1995) and de Palma and Rochat (2000). Other attributes such as environmental awareness, comfort and convenience were found to be statistically insignificant for all mode choice. Apart from that, parking and toll were also found to be statistically insignificant for mode choice car while headway was found to be statistically insignificant for both mode choice bus and rail.

Table 8: MNL results

| Attribute | Coefficient | T - statistic |
|---|---|---|
| Alternative-specific constant for car ($a_1$) | -0.5687 | 0.0048* |
| Income ($a_7$) | -0.1591 | 0.0000* |
| Cost/ Fare ($a_8$) | -0.0729 | 0.0169* |
| In-vehicle time ($a_{11}$) | -0.0079 | 0.0014* |
| Total expenses ($a_{12}$) | -0.0035 | 0.0000* |
| Travel objectives/accessibility ($a_{13}$) | 0.1460 | 0.0014* |
| Security level ($a_{15}$) | 0.1679 | 0.0005* |
| Alternative-specific constant for bus ($a_{18}$) | -0.2197 | 0.2209 |
| Walking distance to station ($a_{20}$) | -0.0011 | 0.0014* |
| **Log likelihood (LL) function** | **-1104.67** | |
| **R-squared ($R^2$)** | **0.4284** | |

* significant at 5 %

The utility functions based on the results in Table 8 were reproduced as follow:

$$U\,(car) = -0.5687 - 0.1591^{*}INCOME - 0.0729^{*}CST - 0.0079^{*}IVT_{car} - 0.0035*EXPRP + 0.1460^{*}OBJ_{car} + 0.1679^{*}SECURITY_{car} \quad (7)$$

$$U\,(bus) = -0.2197 - 0.1591^{*}INCOME - 0.0729^{*}FARE_{bus} - 0.0079^{*}IVT_{bus} + 0.1460^{*}OBJ_{bus} + 0.1679^{*}SECURITY_{bus} - 0.0011^{*}WALK_{bus} \quad (8)$$

$$U\,(rail) = -0.0729*FARE_{rail} - 0.0079^{*}IVT*_{rail} + 0.1460^{*}OBJ_{rail} + 0.1679^{*}SECURITY_{rail} - 0.0011^{*}WALK_{rail} \quad (9)$$

The signs associated with the study variables were correctly representing the utility that an individual choose. People would get out from their mode and look for alternative if the travel time increases. The significance found for walking distance suggests that proper design of public transport stations would encourage people to take public transport service. The signs associated with the monetary related variables reflect that the higher the value the less likely the people continue with their mode of travel. Integrating transportation planning, land use development and communities would effectively change the travel pattern which is found significant (travel objectives with t-value 0.0014).

## 7. VALUE OF TIME ANALYSIS

Apart from the mode choice analysis, analysis to determine the value of time (VOT) was also investigated. An interview survey was conducted and respondents were presented with two sets of SP choice sets with different combinations of factor levels. In this analysis, the effects of time and cost on commuters were examined. Table 9 presents the MNL results from SP survey for those who chose car and bus with reference to the rail mode. Waiting time is not applicable to car mode as car driver can depart at any time.

Table 9: MNL results from SP survey on value of time

| Attribute | Car | | Bus | |
|---|---|---|---|---|
| | coefficient | t-value | coefficient | t-value |
| Constant | 0.3826 | 0.4135 | -1.0714 | 0.0000* |
| Cost | -0.0436 | 0.0021* | -0.1919 | 0.0524* |
| In-vehicle time | -0.01197 | 0.0000* | -0.001725 | 0.7192 |
| Waiting time | - | - | -0.009259 | 0.0568* |

* significant at 0.1

All the signs for the coefficients were found to be correctly representing the situations of which the increase in cost and time are all undesirable. The mode constant for car mode chosen is positive indicating the car is always the preferable mode while bus is less favourable as compared to rail.

In order to determine the effects of time and cost for those travelling with car and bus with the reference to rail, VOT was computed by dividing each time parameter by the cost. The results were presented in Table 10.

Table 10: Monetary valuations form the SP mode choice

| Value of Time | Car | Bus |
|---|---|---|
| Value of in-vehicle time in RM/hour ($VOT_{in\text{-}vehicle}$) | 16.20 | 0.54 |
| Value of waiting time in RM/hour ($VOT_{waiting}$) | - | 2.88 |

Based on the results obtained in Table 10, value of in-vehicle time, $VOT_{in\text{-}vehicle}$ for mode choice car is very high as compared to the bus mode. This indicates that those who prefer cars were more concerned with the time spend in their cars as compared to those who prefer bus. This is true as those who commute using private vehicles are more sensitive to the time and are willing to pay more in order to arrive on time. As for the VOT for bus, the value of waiting time, $VOT_{waiting}$ is higher than the value of in-vehicle time, $VOT_{in\text{-}vehicle}$. This is a fairly typical result as a reduction in waiting is seen to be of higher importance than reduction in in-vehicle time. This is because bus users tend to get anxious and upset if they spent a long time waiting for the bus to arrive but not so if they have to spend a long time in the bus.

## 8. CONCLUSION

In this paper, the mode choice (car, bus and rail) behaviour in Klang Valley was examined through stated preference approach. Attributes studied were identified based on the concept of sustainable transportation. Travel objectives or accessibility and walking distance to public transport services were found significant in modal split choice and this suggest the importance of integration design of transportation plan, communities and land use planning. Travel time and travel cost results are in consistent with other findings with their influence in mode choice.

## REFERENCES


Andriantiatsaholiniaina, L. A., Kouikoglou, V. S. and Phillis, Y. A. Evaluating strategies for sustainable development: fuzzy logic reasoning and sensitivity analysis. Ecological Economics, 48, pp. 149 – 172. 2004.

Banister, D. Sustainable urban development and transport – A Eurovision for 2020. Transport Reviews, Vol. 20, No. 1, pp. 113 – 130. 2000.

Barbier, E.B. The concept of sustainable economic development. Environment Conservation, 14, pp. 101 – 110. 1987.

Barter, P. Malaysia urban transport in an Asian and global context: Challenges of car and motorcycle domination. *Sarawak Development Journal*, Vol. 2, Issue 1. 1999.

Bonsall, P., Beale, J., Paulley, N. and Pedler, A. The differing perspectives of road users and service providers. Transport Policy, 12, pp. 334 – 344. 2005.

CfIT. Public attitudes to transport in England. Report available on http://www.cfit.gov.uk/reports/mori2001/pdf/mori2001.pdf. 2001.

CHKL (City Hall Kuala Lumpur). Kuala Lumpur structure plan 2020. Available online at http://www.dbkl.gov.my/pskl2020/english/index.htm (accessed March 2007). 2003.

Common, M. and Perrings, C. Towards an ecological economics of sustainability. Columbia University Press, New York, p. 525. 1992.

de Palma, A. and Rochat, D. Mode choices for trips to work in Geneva: an empirical analysis. Journal of Transport Geography 8, 43-51. 2000.

Downey, M.L. Transportation trends. Paper presented at the Symposium on Challenges and Opportunities for Global Transportation in the 21st Century, Cambridge, M.A. 1995.

EFTE (European Federation For Transport and The Environment), 1994. Green urban transport: A survey. Preliminary Report 94/2, January. 1994.

Gilbert, R. and Tanguay, H. Sustainable transportation performance indicators project. The Centre for Sustainable Transportation. www.cstctd.org. 2000.

Hensher, D.A., Rose, J.M. and Greene, W.H., 2005. Applied choice analysis – A primer. Cambridge University Press. 2005.

Kiggundu, A. T. Revitalizing public transport in the Greater Kuala Lumpur, Malaysia: Issues, challenges and prospects. Paper presented at Internal Seminar by Department of Geography, Faculty of Arts and Social Sciences, University of Malaya, Kuala Lumpur, Malaysia. 2005.

Linquist, E. Moving toward sustainability: Transforming a comprehensive land use and transportation plan. Transportation Research Record 1617. 1998.

Marjan, J. M., Radin Sohadi, R. U. and Ahmad, S. K. Modelling accidents along two lane single carriageway roads in Malaysia- Research methodology & data collection. Seventh Malaysia Road Conference. 17-19 July 2007, Sunway, Malaysia. 2007.

Montgomery, D.C. Design and analysis of experiments. Fifth Edition. Arizona State University, John Wiley & Sons Inc. 2000.

NRTEE (National Round Table on the Environment and Economy). Sustainable transportation in Canada: Backgrounder. Ottawa: NRTEE. 1997.

OECD. Motor vehicle pollution: Reduction strategies beyond 2010. 1995.

Osula, D. O. A. A procedure for estimating transit subsidization requirements for developing countries. Transportation Research A, 32 (8), pp. 599 – 606. 1998.

Pearce, D.W., Markandya, A. and Barbier, E.B. Blueprint for a green economy. Earthscan, London, p. 192. 1989.

Pendyala, R., Kostyniuk, L. and Goulias, K. A repeated cross-sectional evaluation of car ownership. Transportation 22, 165 – 184. 1995.

Sinha, K. C. Sustainability and urban public transportation. Journal of Transportation Engineering @ASCE, July/August. 2003

The Star. Putting the best route forward. Article on press dated 24 September 2006.

WCDE (World Commission on Environment and Development). Our common future. Oxford University Press, Oxford, p. 400.

WHO (World Health Organization). Creating Healthy Cities in the 21st Century. Background paper prepared for the Dialogue on Health in Human Settlements for Habitat II. 1997.

Yahya, N. Z. and Sadullah, A. F. M. Atmospheric pollution (CO) from vehicle emissions: A comparison between an urban freeway and urban intersection at different surrounding. Malaysian Universities Transport Research Forum (MUTRF). 2002.

Bus

Ziestman, J. and Rilett, L. R. Using TRANSIMS and ITS data to quantify aspects of sustainable transportation. Proceedings of the First IGERT Graduate Student Conference, Held at the University of California, Davis. 2000.

Zietsman, J., Rilett, L. R and Kim, S. J. Sustainable transportation performance measures for developing communities. Texas Transportation Institute Texas A&M University System, College Station, Texas 77843-3135. 2003.

# LITERATURE REVIEWS COMPARISON ESSAY ON TRAVEL FOCUSING ON TRAVEL NEED, TRAVEL BEHAVIOUR AND TRAVEL PATTERN.

**Diana MOHAMAD**
*School of Natural and Built Environments*
*University Of South Australia*
*SA 5000 Adelaide*
*Diana.Mohamad@postgrads.unisa.edu.au*

**Syahriah BACHOK**
*Transport System Centre*
*University Of South Australia*
*SA 5000 Adelaide*
*Syahriah.Bachok@postgrads.unisa.edu.au*

**ABSTRACT**

Understanding the previous, present and to date research studies on travel in terms of factors that influence such phenomenon as well as response to changes in circumstances that matters are an important component in assessing and projecting future travel-related issues and matters. Defined as 'trip or multi-trips made from origin to destination' or 'moving any distance by any means of transportation' (Mokhtarian, Salomon and Redmond, 2001), travel whether in the form of vehicle mode or non-vehicle mode, has always been part of human daily activity. Timmermans et. al (2003) well thought-out travel into two positions namely (1) trip which defined as movement between two activities carried out at different locations and (2) tour which defined as a sequence of trips where it starts and ends at the same location. To date, studies focusing on travel have it classified and categorized travel into different groups depending on the travel-related field studied as well as by the breadth of the domains which they are likely to affect. This paper attempted to draw some conclusion with regard how a certain parameter influences one's travel in terms of travel needs, travel behaviors as well as travel patterns. Not to mention that pervasiveness of habits, personal circumstances and other factors takes place over time, some of the studies presented provide evidence that a certain parameter works uniquely on two different individual.

Keywords : travel behavior, travel need, travel pattern, demography, urban form.

## 1. INTRODUCTION

Defined as 'trip or multi-trips made from origin to destination' or 'moving any distance by any means of transportation' (Mokhtarian, Salomon and Redmond, 2001), travel whether in the form of vehicle mode or non-vehicle mode, has always been part of human daily activity. Timmermans et. al (2003) well thought-out travel into two positions namely (1) trip which defined as movement between two activities carried out at different locations and (2) tour which defined as a sequence of trips where it starts and ends at the same location. To date, studies focusing on travel have categorized travel into different groups depending on the travel-related field studied. Literature reviews of travel studies (travel studies typology) can be summarized and organized in any number of ways for example, by travel mode (private transportation, public transportation, walking, cycling, etc), by travel purpose (work-related trip, entertainment-related trip, emergency-related trip, trip chains, etc), by travel demand (aging population, vehicle ownership, income level, etc), travel studies analytical methods (regression, cross tabulation, simulation, etc) or urban forms' measures and characteristics (street pattern, accessibility, urban density, etc). In addition, travel researches can suitably be assorted into groups of travel behavior or travel pattern (see Crane, 2000 and Morikawa, 2001).

The purpose of this article is to provide an overview of this foundation and to draw conclusion with regard how a certain parameter influences one's travel in terms of travel needs, travel behaviors as well as travel patterns. Not to mention that pervasiveness of habits, personal circumstances and other factors

takes place over time, some of the studies presented provide evidence that a certain parameter works uniquely on two different individual. The following section of this paper consists of literature review where some of the studies selected will illustrate travel practice and to review the results of the studies and ends with conclusion where this section serves the purpose of giving some insights and a better ground for the future research.

## 2. LITERATURE COMPARISON

According to Mokhtarian, Salomon and Redmond (2001), travel is not purely derived. Simply put, travel is made not only because there is a need for travel or for the sake of getting to the destination. Travel has value of its own and also an intrinsic positive utility that is to say that, a certain positive factors or characteristics that lead some people to engage in travel might not be working on the same condition to the others. This suggestion is much similar with the fact pointed by Michel de Montaigne (1957), Robert Louis Stevenson (1879) and Bruce Chatwin (1989) where in these literature reviews, travel is described as 'an activity which man took pleasure in doing it. People love to travel for the sake of travel itself where there are times when travel itself is the desired activity' (as cited by Mokhtarian, Salomon and Redmond, 2001).

By examining travel's 11 most measured variables, this study was carried out to serve the intention of having clear policy implications. Results show that rather than determined by demographically-based needs (age, income, gender, level of education), travel is much more influenced by traveler's attitude toward travel. Additionally, findings also pointed out those demographically-based needs are positively related to travel engagement when subjective variables such as travel liking, the adventure-seeker personality, the travel stress attitude as well as the excess travel indicator were evaluated jointly. Study concluded that the intention of reducing vehicle miles traveled (VMT) will have to depend on the traveler's desirability whether they desire more or less mobility than they currently experienced. Much similar to the conclusion, Sperling (1995) in Future Drive : Electric Vehicles and Sustainable Transportation (as cited by Simich and Gilbert, 1998) wrote that people stand firm with better technology innovation for benign transportation system approach over behavior 'reconstruction' and changes.

Just like 'cars will now have to adapt to the city' approach adopted by Amsterdam (The New York Times, 1993 as cited by Simich and Gilbert, 1998); other certain approaches for example increases road pricing, limited parking space and parking time proved to be insufficient to assist travel demand (Crane, 2000 and Mohamad, unpublished). Other than that, Kain and Fauth (1977) provide evidence that even though urban form does play a role in influencing travel engagements, people tend to travel once they have access to cars (as cited by Crane 2000). Sperling (1995) in Future Drive : Electric Vehicles and Sustainable Transportation (as cited by Simich and Gilbert, 1998) noted down that affection to cars derives from the unparalleled freedom, privacy, security and convenience that cars provide (see also Mohamad and T. Kiggundu, 2007).

Seeing travel from different angles, some transportation scholars are opposed to Mokhtarian, Salomon and Redmond (2001). Attempt to seek the academics' opinion on telecommuting working arrangement, Mohamad (unpublished) indicated that if travel time and travel cost is to increase by 50%, people will tend to save-up and divert private travel needs by engaging in another transportation mode (see also Shalom Reichman 1976 and Peter Jones 1978 as cited by Mokhtarian, Salomon and Redmond, 2001). Though holds its own value, travel is made not just for the sake of travel itself. Purpose, fuel price and other factors are calculated before the trip is made (Centre for Sustainable Transportation, 1998). As stated earlier, looking from the perspective of urban form, significant influence is detected where urban density, street pattern as well as pedestrian facilities play a role in 'manipulating' the travel pattern (see Susilo and Kitamura 2006; and Crane Randall 2000 as cited in Ewing 1977; Stone, Foster and Johnson 1992; Giuliano and Small 1993; McNally and Ryan 1993; Rabiega and Howe 1994). Rutherford, McCormack and Wilkison (1996) who concerned and engrossed with research question 'to what extent will behavior corresponds to various land use and design characteristics' attempted to summarize actual travel behavior. Although urban density can be replicated, this study supported the idea of household's travel amount can be reduced by mixed-use development (as cited by Crane Randall 2000).

Susilo and Kitamura (2006) provide the contrary evidence to Cohen and Kocis (1980), Kollo and Purvis (1984) and Levinson and Kumar (1994). Based on behavioral hypotheses 'worker with longer commute tend to visit locations on the way to and from work' and 'worker with shorter commute tend to make a separate home-based trip chain after work', this study attempted to draw conclusion with regard structural changes in Japanese commuters' daily travel. Great transit terminals access in Osaka Metropolitan and longer commute explain the increase in the number of trip chains and decrease in time-spent for non-work activities for both transit commuters and auto commuters. Transit commuters in Osaka metropolitan, if to compare with auto commuters, were more mobile and had higher level of activity engagement. Moreover, the results show that transit commuter had shorter commute time than auto commuter due to high level of services (access) provided at the transit terminals. Thus, adopted hypotheses and conventional wisdom 'automobile is suited for trip chaining and auto commuter tend to chain trips' proved to be not suitable for all travel environment.

Morikawa (2001) generally agrees with conventional wisdom that increase in income level, low level of public transportation services, traveler's attitudes toward vehicle, and vehicle ownership (see also Mohamad and T. Kiggundu, 2007) are factors that manipulate travel behavior. Cohen and Kocis (1980), Kollo and Purvis (1984) and Levinson and Kumar (1994) who share the same assumption suggested that the trend of motorization and suburbanization has significantly influenced the way people traveled due to the flexibilities in auto travel which allow auto commuter to adapt to changes in the travel environment (as cited by Susilo and Kitamura, 2006). Morikawa (2001) showed that of those factors, insufficient public transportation services paired with increase in income level play a role in determining travel behavior in Kuala Lumpur.

Information presented one similarity between Bangkok, Kuala Lumpur and Manila where male travelers are significantly positive related to cars and motorcycles, unlike Nagoya's, which prefer cars to motorcycles. Nevertheless, higher public transportation modal share both in Bangkok and Manila than Kuala Lumpur explain the age variables where aged travelers' in Bangkok and Manila prefer public transportation to make trips. Black (2001) agreed that to a certain extent, public transportation serves as the social service for the aging population where this statement does imply that public transportation with required standards are needed by the targeted group. However, this does not support the fact that increases in an aging population will result in increases in bus-transit ridership. From his study research with regard to bus-transit ridership of the aging population in US, Black came up with clear notion that 'this trend is continuing today and while it will not eliminate the need for bus transit, it is suggestive that the mode will never again reach parity it held with motor vehicles'.

Doted on hypothesis 'new urbanism which exercising transit-oriented development will have people to ease automobile use significantly', Nelson and Niles (1999) sought to understand the relationship between market and non-work travel pattern through market dynamics in order to draw an insight with regard how commercial centers might be configured to maximize transportation benefits. Five consumer behavioral factors that influence non-work travel pattern were listed, namely: bargain hunting, comparison-shopping, preference for variety, location and schedule flexibility. Nelson and Niles (1999) proved that resistance occurs from local residents when changes in land-use are to be implemented. Moreover, research shares the same results as Downs (1994) where a certain number of commercial center transit-oriented based developments needed to serve targeted area sufficiently (as cited by Nelson and Niles, 1999) and also compact, mixed use commercial centers come together with complex trip patterns. Results of Nelson and Niles (1999), Sperling (1995, as cited by Simich and Gilbert, 1998) and Mohamad and T. Kiggundu (2007) are complementing each other in a way that all these studies generally agreed with 'people do values cars' hypothesis.

Using number of trips; number of tours and number of stops as research indicators and definition of travel as mentioned earlier, Timmermans et al (2003) did a comparison study with regard to what extent does spatial context influence travel behavior as well as sought whether travel patterns follow their own regularities, if any. Trip-related and tour-related questions (number of daily home-based trip/tour, number of daily trip/tour per person, average daily trip/tour ratio – data collected at day of the week level) revolving

around spatial context, households' types and transport mode used steered the study. Comparing United States, Japan, Canada, United Kingdom and Netherlands, study found the following results :

1. Japan (Fukuoka) results in significantly low number of daily home-based tours while United States (Portland) and Canadian metros show the opposite. [In Fukuoka case, study shares the same results as Susilo and Kitamura (2006) where high quality of public transportation services did influence traveler's trip chain patterns].
2. Number of home-based tours decreases if public transport is used.
3. Number of home-based tours does not seem to be manipulated by spatial setting and mode of transportation. [In a way, study provided the same results as Kain and Fauth 1976; Messenger and Ewing 1996 and O'Regan and Quigley 1998 (as cited by Crane 2000), however, one difference spotted where mode of transportation does seem to influence travel in these studies].
4. Male traveler tends to make single-stop trip compared to female traveler who tend to involve in trip-chaining. [Information complemented Kumar and Levinson (1995 – as cited by Timmermans et. al 2003) findings where women make significantly more trips than average number of home-based trips indicator].

It is widely known that transportation level of service, distance from home to workplace as well as socioeconomic issues significantly influence traveler's travel-related decisions. Study conducted by Hamed and Olaywah (1999) set to examine commuters' of private vehicle, bus and servis taxi travel-related decisions in Amman, Jordan with strong interest toward morning departure time to the workplace and types of after work activities. Information indicates that morning departure time to the workplace varies with difference factor has unique influence to different targeted group of commuters. Private car commuters seem to be influenced with presence of children factor while distance to service stop from home, age, gender as well as waiting time are the factors influencing bus and taxi commuters' morning departure time. Distance to the workplace has significant impact on all targeted groups where morning departure time is coherent with distance to the workplace, that is to say this information indirectly provide some insights about transportation system and management of the study area which dominated by the private transportation.

As for the types of after work activities (categorized into groups of household errands, home maintenance and business-oriented activities) Hamed and Olaywah (1999) noted that though there is no significant difference in targeted groups' departure time from work, this variable, on top of distance to the activity, mode of transportation together with whether the commuter's spouse is employed or not factors did influence the after work activities exercised. Provided with the unparallel accessibility, unlike bus and servis taxi commuters, private cars commuters tend to make home-based trip for a wide range of after work activities particularly household errands. Though the study generally agrees that most of the trips made were contributed by the private vehicles, however, if the distance to the activity increases, private cars commuters as well as bus and servis taxi commuters are less likely to perform the activity. In a way, this is slightly differs from Kain and Fauth (1977).

Results of a study by Lee and McNally (2003) addressing the structure of weekly activity/travel patterns show that travels of short and long duration produce different patterns with regard to types of activities (work events – work and school; and non-work events – maintenance, shopping/services, recreation/entertainment and social). Defined as sequence of out-of-home stops, travel's tour is divided into two categories namely tour with single stop (multi-activities take place at one particular stop) and tour with non-single stop (activities occur at different places). Using an almost similar method as Doherty's and Miller's (2000, as cited by Lee and McNally 2003) realistic planning conditions to observe household's behavior scheduling with regard to travel planning, Lee and McNally (2003) found that structured travel (before week planning and within week planning) tend to be activities with long duration (both tour with single stop and tour with non-single stops) while opportunistic travel (within day planning and spontaneous planning) tend to be activities with short duration. Findings supported Cullen's and Godson's (1975, as

cited by Lee and McNally 2003) activity-peg proposition where certain activity occurs in daily schedule acts as 'pegs' around with others activities are planned and scheduled earlier.

Other than that, decision for visiting places is not necessarily has to be pre-determined, it is more likely to be impromptu decision with higher propensity increased as stop sequence increased. However, the proportion of opportunistic stops decreases as travel time increases. The former results are much similar to Svenson's (1989) and Chen's (2001) findings with regard activity schedules are often 'incomplete'. In addition, findings also indicated that though being planned beforehand, in-home activities tend to be improvised from time to time and people do not intentionally schedule non-work events unless the duration is of a certain length. Moreover, when analyzing maintenance; recreation and shopping; study found out that none of these three factors is expected to be stand alone independently. That is to say that each of three events are planned and scheduled depending on the other two events.

Interestingly, Black (2001) in 'An Unpopular Essay on Transportation' sought the clear conclusion with regard to several popular notions which exist in the transportation and transport geography literature. As some of the developing countries believe in the 'globalization and traffic flows' approach to overcome traffic congestion (see Zakaria 2003), Black proved that globalization is all about responses to some economic incentive such as low wages. Sharing much similar assumption is Gabel (1994 as cited in Black 2001) where he quoted that '…As trade extends in geographical reach, transportation rises as a proportion on international production. And although the world economy is now highly integrated, further liberalization of international trade can be expected to increase transport inputs per unit of international product, *ceteris paribus*'.

Just like 'globalization and traffic flows' approach, 'construction of the missing links and networks' approach deemed to be one of the workable approaches to alleviate traffic congestion as well as other transportation issues (see also Eighth Malaysian Plan 2001-2005 Report and Kuala Lumpur Structure Plan 2020 : Transportation). Seeing this from different point of view, Black considered this approach nothing more than 'the new network will do little to stimulate the economics of Europe remote areas'. Black supported this statement with illustration of the relationship between average path length and network length where Black firmly stated that 'as a network's length increases, there are very few gains in terms of reducing the average length of the shortest path and at some point the gains may cease prior to complete connectivity'. Black concluded that geographic area covered by the network has to be changed if one's relative location is to be upgraded. In a way, this notion is similar to the concept of distance enterprise and transborder telecommuting work arrangement (Herman Miller 2001, Vittorio Di Martino 2001 and Derrick J.Neufield and Yulin Fang 2004).

Sought to understand car travel from the economy perspective, Dargay (2006) focuses on the effect of prices and income on car travel in the United Kingdom. Based on the fact that some factors (pervasiveness of habits, inertia, imperfect information) takes place over time to influence car travel and the response to changes in circumstances (income, personal circumstances, transport supply and prices) does not occur instantaneously, Dargay tried to draw conclusion with regard the factors determining household car travel especially the effects of the abovementioned parameters (issues). By using pseudo-panel data (one of the repeated cross-section data methods) which is of longer time periods than panel survey and providing more detailed information on individual behaviour and circumstances, Dargay attempted to analyse household car travel by investigating the effects of fuel prices, vehicle fuel efficiency and income on car travel. Assuming that, on average, fuel prices and fuel efficiency are the same at any point throughout the study period, study found that:

1. Total expenditures increase up until the household's head reaches late 40s and decline thereafter. This is also largely depends on the number of adults member in one's households where an addition adult increases the household's car travel which will lead to increase in total expenditures.
2. Income has a significant positive influence where car travel responds more strongly to rising incomes than it does to falling income. That is to say that when the income decreases, car level use fall slightly than normal standard.

3. The level of car travel is said to increase even during the period of fallen income and will continue to do so until saturation is reached. Study noted that this is largely contributed by habits and personal circumstances factors where people tend to opt for technology which will satisfy their need best (see also Simich and Gilbert, 1998).

It is also noted that income has a significant effect on use per car where the use level decreases when income falls and vice versa. Further, costs of ownership and motor fuel have negative effect on car travel where when prices of these two components increases, the level of car travel decrease. In contrast to Holtzclaw's studies (1990 and 1994 as cited by Handy, 1996) where 'modest difference in income between neighborhoods should make no difference in the analysis' as well as 'average income proved to have an insignificant relationship with either auto ownership or total automobile travel', Dargay (2007) concluded that as the next generation secured a certain level of stable income and slowly grow into successive cohort than the previous generation; motoring has become more prevalent and widespread.

To date, studies on the link between urban form and travel behavior results in a number of findings. Handy (1996) presented some of the previous studies' results with regard land-use impacts on travel behavior. A comparison of travel patterns between different neighborhoods by Friedman et al (1994) results in 'higher percentages of transit and other non-automobile use in traditional neighborhoods' but on the other hand, Cervero and Gorham study (1995) confirmed that transit neighborhoods have a higher use of transit. Frank and Pivo (1994) provide evidence that urban form variables were significant for predictors of travel patterns and different aspects of urban form were significant for different types of trips. This study is supported by LUTRAQ 1993 findings where results demonstrated as the pedestrian environmental factor (PEF) increases, a significant decline detected in vehicle miles travelled and at the same time an increase in the percent of non-automobile mode shares. Furthermore, LUTRAQ study indicated that 'pedestrian-friendly neighborhood which surrounded by automobile-oriented suburban development, do not generate the same level of non-automobile use as pedestrian-friendly neighborhood surrounded by other pedestrian-friendly neighborhoods'.

## 3. CONCLUSION

'Travel is not purely derived' hypothesis can be validated if the same choice(s) will be made in the same situation(s) given that Hauser (1978 pg 409 as cited by Mokhtarian and Bagley, 2000) points out '…if individuals make repeated choices and do not always select the same alternative, then $p^2 = 1$ is not possible'. A finding of Rutherford's, McCormack's and Wilkinson's study (1996) indirectly holds the conventional wisdom 'traveler's attitude does affect travel'. Same results can be found through Mokhtarian, Salomon and Redmond (2001) as well as Mohamad and T. Kiggundu (2007). As major percentage of travel trip made is derived from private vehicle, questions like 'assuming there are no work-related constraints and non work-related constraints, how much would you like to travel and by what means' and level of opportunity accessibility-related questions for example the total number of relevant opportunities, the spatial distribution of opportunities, individual's spatial location and individual's ability to overcome spatial separation will help to a broader picture and a better ground of understanding the travel.

## PRIMARY REFERENCES

Black, W.R (2001). *An Unpopular Essay On Transportation*. Journal of Transport Geography 9 (2001) 1-11. Elsevier Science Ltd.

Dargay, J (2007). *The Effect of Prices and Income on Car Travel in the UK*. Transportation Research Part A 41 (2007) 949-960. Elsevier Ltd.

Derrick J.Neufield and Yulin Fang (2004). *Predicting Telecommuter Productivity*. Proceedings of the 37th Hawaian International Conference on Systems Science.

Eighth Malaysian Plan 2001-2005 Report (2006). *Infrastructure and Utilities.* Economics Planning Unit, Prime Minister Department.

Hamed, M.M and Olaywah, H (1999). *Travel-Related Decisions By Bus, Servis Taxi And Private Car Commuters In The City Of Amman, Jordan*. Jordan University of Science and Technology.

Handy, S (1996). *Methodologies for Exploring The Link Between Urban Form And Travel Behavior*. Transpn Res.-D, Vol. 1 No. 2 pp 151-165. Elsevier Science Ltd.

Herman Miller (2001). *Telecommuting: Working Off-Site*. Herman Miller Inc

Morikawa, T (2001). *Travel Behavior Analysis and its Implicationto Urban Transport Planning for Asian Countries* : *Case Studies of Bangkok, Kuala Lumpur, Manila and Nagoya*. ICRA Project Report.

Lee, M.S and McNally, M.G (2003). *On The Structure Of Weekly Activity Activity/Travel Patterns*. Transportation Research Part A 37.

Mohamad, Jamilah and T. Kiggundu Amin (2007). *The Rise Of The Private Car In Kuala Lumpur, Malaysia- Assessing the Policy Options.* IATSS Research Vol. 31 No 1.

Nelson, D. and Niles, J.S (1999). *Market Dynamics And Non-Work Travel Pattern : Obstacles To Transit-Oriented Development*. Transportation Research Board.

Ninth Malaysia Plan 2006-2010 Report (2006). *Mainstreaming Information and Communications Technology*. Economics Planning Unit, Prime Minister Department.

Ninth Malaysia Plan 2006-2010 Report (2006). *Promoting Environmental Stewardship*. Economics Planning Unit, Prime Minister Department.

Randall, C (2000). *The Influence Of Urban Form On Travel : An Interpretive Review*. Journal of Planning Literature, vol. 15 issue 1, pg3 21p.

Rebecca Elmore-Yalch (2004). *2004 Transit Needs Study*. Final Report Capital Area Transportation Authority in Partnership with Eaton County Transportation Authority and Delta Township. Northwest Researsch Group, Inc.

Susilo, Y. O and Kitamura, R (2006). Structural Changes in Commuters' Daily Travel : The case of Auto and Transit Commuters in the Osaka Metropolitan Area of Japan, 1980-2000. Transportation Research Part A 42 (2008) 95-115. Elsevier Ltd.

Vittorio Di Martino (2001). *The High Road To Telecommuting*. Geneva.

Zakaria, Zulina (July 2004). *Framework for Designing Regional Planning Architecture for APTS-Enabled Regional Multimedia Public Tranportation System*. Massachusetts Institute of Technology.

**SECONDARY REFERENCES**

Ab. Rahman (undated). *Transport and Communication for Urban Development Car Poling in Kuala Lumpur Public Perception*. University Technology of Malaysia.

Barter P.A. *Transport Dilemmas In Dense Urban Areas : Examples from Eastern Asia.* Sustainable Urban Forms for Developing Countries, pp. 271-284 (London Press).

Barter P. A. *Transport, Urban Structure and lock-in in The Kuala Lumpur Metropolitan Area.* International Development Review, 26 (1), 1-24.

Choocharukul, K; Van, H.T and Fujii, S (2008). *Psycological Effects of Travel Behavior on Preference of Residential Location Choice.* Transportation Research Part A 42 (2008) 116-124. Elsevier Ltd.

# APPLICATION OF PCE VALUES IN CAPACITY AND LOS ANALYSIS

**Johnnie Ben-Edigbe**
*Associate Professor of Civil Engineering*
*University of Technology Malaysia*
*81310 Skudai, Johor*
*Malaysia*
*Email: edigbe@utm.my*

## ABSTRACT

Roadway capacity is the maximum flow reached at critical density when quantified and is the maximum flow reached at optimum speed when measured qualitatively in LOS analysis. PCE could be construed as a function of (independent variables) traffic flow, heavy vehicles, roadway, traffic and ambient conditions. The conditions vary singularly or in combination from road section to section or point to point, probably explaining why PCE values are merely approximations of the effects of vehicle-mix. The study is intended to show that PCE values vary relative to prevailing conditions hence should not be taken as fixed values except in the unlikely situation of homogeneous traffic stream. The paper explored the effects of poor level terrain on PCE values under daylight and clear weather conditions. Standard PCE values were compared with simulated empirical PCE values obtained from 6 sites in Malaysia using headway method. Based on the hypothesis that PCE values differ significantly between roadway with and without vertical deflections; the study found substantial changes in PCE values and concluded that, ignoring PCE modifications could lead to grossly inaccurate capacity and level of service estimate with attendant consequences for traffic and transportation modelling.

## 1. INTRODUCTION

The road network is a maze of links and intersections, and each with its own physical characteristics that influence traffic flow under prevailing conditions. Highway Capacity Manual - HCM special- report 209 (*4*) defines highway capacity as 'the maximum hourly rate at which vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given time period under prevailing roadway, traffic and control conditions'. The definition recognises and rightly so, the potential for substantial variations in flow during an hour, and focuses analysis on intervals of maximum flow. However, the definition missed vital elements (maximum flow and critical density) that define highway capacity and focuses on level of service based capacity (maximum flow and optimum speed). In any case the definition suggests that highway capacity can be a point of section measurement, which is quite interesting and valid, because highway capacity is a point or section measurement under prevailing conditions.

Since highway capacities vary from point to point, section to section, it can be argued that applied passenger equivalent (PCE) values in the capacity and level of service analysis would also vary. Passenger car equivalent values are measures of vehicle performances. Their use is central to highway capacity and LOS analysis where mixed traffic streams are present. By applying PCE values in highway capacity and level of service (LOS) analysis, in essence attempts are made to determine the number of passenger cars displaced in the traffic flow by non passenger cars like motorcycle, light goods vehicles, buses, trucks, and heavy goods vehicles under prevailing roadway, traffic and ambient conditions. This is to be expected because qualitative and quantitative measurements of traffic flow are constrained by prevailing conditions. Thus, the assertion that PCE values are somehow fixed for terrain type, road geometry, directional distribution, and traffic conditions is somehow faulty because road, traffic and ambient conditions are known factors that affect highway capacity it can be argued.

Overtime scholars have argued about the definition and bases for numerical derivation of PCE values. In fact many researchers have tried to quantify the effect of non-passenger cars on traffic flow relying on HCM approach but using different methodologies and equivalency criteria. A few of those studies utilized field data, while most employed traffic simulation to derive PCEs for a wide range of conditions often with doubtful and exaggerated outcomes.

However, the paper is not about derivation of PCE values, rather it focuses on the correct application of dynamic PCE values in highway capacity and LOS analysis given poor road surfacing conditions. It explored the effects of poor level terrain on PCE dynamic values under daylight and clear weather conditions. In the paper, a simplistic headway method was used to modify standard Malaysia PCE values for highway capacity and level of service analysis using empirical data from 6 field study sites.

Malaysia's road system is extensive and is among the finest in Asia. It covers a distance of 63,445 km with about 5.2 million total numbers of registered vehicles at the end of 1990. The inter-urban North-South Expressway, New Klang Valley Expressway (NKVE) and the Federal Highway Route 2 (FHR2) are the largest road transportation infrastructure in Peninsula Malaysia. The 848 km expressway links major industrial areas to urban centres commencing from Bukit Kayu Hitam to Johor Bahru. The East-West Highway serves as part of the Asian Highway System linking Thailand with Malaysia. In 1993, there were 5.4 million motor vehicles of which 38.6% were motorcars, 54.8% motorcycles and 6.6% goods vehicles. This increased by 50.5% to reach 8.1 million motor vehicles in 1997, with motorcycles accounting for 53% of the total, followed by motorcars 40% and goods vehicles 7%. In other to improve the efficiency and effectiveness of road services, application of PCE in capacity analysis must be modified to reflect prevailing conditions.

## 2. QUANTITATIVE ASSESSMENT OF ROAD TRAFFIC OPERATION

For the purpose of measuring *quantity*, the parameters, density and flow are important, because density is a finite element that describes the number of vehicles per unit length of the road. Flow which is the number of vehicles passing a given point on or road section per unit time measures the traffic stream quantity and traffic flow demand. In essence, quantitative assessment of road traffic operation can be construed as traffic flow and capacity measurements. Estimating highway capacity is not without problems, according to Minderhoud *et al (7),* the problem consists of a series of essential points of interest that include among others; Type of Data To Be Collected, Location Choice for Observations, Choice for Appropriate Averaging Interval, Needed Observation Period, Required Traffic State, and Lane.

Capacity-estimation problem can be divided into two categories: the direct-empirical studies and indirect-empirical methods noting that all the methods are a mixture of observation and theory. It could be argued that some methods have more theoretical justification than others especially those that have to contend with probabilistic functions. For example the basic principle capacity estimation using headway models is deterministic, relying on parameters of the compound probability density function of headways with results that may not bear resemblance to the practical value. Consider equations 1 and 2 below.

$$q = uk \tag{1}$$

$$q = -\beta_o + \beta_1 k - \beta_2 k^2 \tag{2}$$

Where; q is flow, u is speed, k is density, $\beta_o$ is a constant, $\beta_1$ and $\beta_2$ are coefficients; $\beta_o$ describes roadway density, $\beta_1 k$ describes vehicle speed while $\beta_2 k^2$ describes traffic flow.

In order not to violate the rules of concavity, traffic flow rates are constrained within finite density boundaries (0 and $k_j$) as contained in many literatures. Whether traffic is operating at congested or uncongested state, the operations are contained with the finite density boundaries. These finite boundaries cannot be exceeded, also the critical density ($k_c$) cannot be $k_j$ divided by 2 because the concavity is not symmetrical, speeds are not the same for the congested and uncongested sectors (see Figure 1).

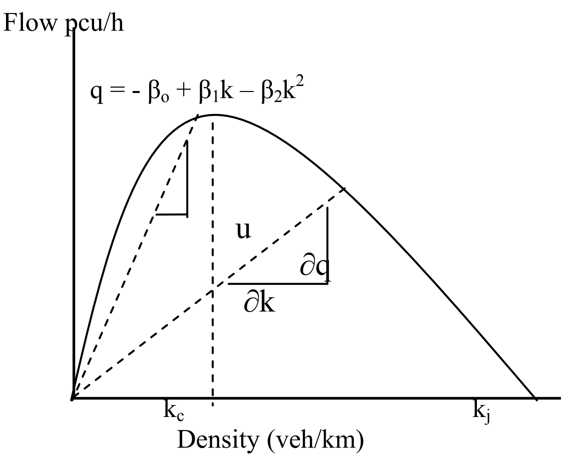



Figure 1 Flow – Density Curves

## 3. QUALITATIVE MEASUREMENT OF ROAD SERVICE

Road network is a maze of nodes (intersections) and links; each with its own physical characteristics that influence prevailing traffic flows from point to point, section to section. Travel time is what most road users are concerned about. It is a useful guide for measuring the effectiveness of roads; when used in conjunction with delay and capacity utilisation; the quality of service can be assessed. As the quality of service decreases, so will the average travel speed, drivers will experience more delays, platooning becomes intense as density increases. That being the case, it can be asserted that the mixed-traffic stream harmoniser (PCE) would also vary relative to the prevailing road, traffic and ambient conditions. Consider equation 3 below,

$$u = -\,{}^{\beta o}/_{k} + \beta_1 - \beta_2 k \qquad (3)$$

Because ${}^{\beta o}/_{k}$ is small and negligible equation 3 can be rewritten as; $u = \beta_1 - \beta_2 k$ (4)

Let, $k = {}^{q}/_{u}$; $\beta 1 = u_f$ and $\beta_2 = \frac{\partial_u}{\partial_q}$

For the uncongested traffic state ($u_u$); $u_u = u_f - \left[\frac{\partial_u}{\partial_q}\right] . \frac{q}{u}$ (5)

For the congested traffic state ($u_c$); $u_c = \int_0^{qm} \frac{\partial_u}{\partial_q} q$ (6)

In qualitative measurement of road service, speed is a function of flow, equation 5 is a negative linearity curve with maximum ($u_f$) and optimum ($u_0$) speeds as extreme values, so it's only applicable to the uncongested portion of the speed-flow curve in figure 2 below. The linear function is also constrained by free flow speed ($u_f$) and maximum traffic flow ($q_m$) boundaries if linear rules are not to be violated. Within this section, highway traffic operates in uncongested flow mode with speed oscillating between $u_f$ and $u_0$. However, once absolute highway capacity is reached at optimum speed, additional vehicles in the traffic stream will trigger congested traffic flow mode where vehicle speeds become unpredictable. Whilst in uncongested mode, speed-flow curve oscillates between free flow speed and optimum speed; once the optimum speed is reached and surpassed perturbation sets in, oscillation movement stops; the positive linear curve goes into a coil and recoil mode. In any case, should highway prevailing conditions improve, traffic disturbances removed, and optimum speed reached then the curve balloons back to optimum speed and starts to oscillate again. The sequence can be repeated many times especially in urban areas and city centres.

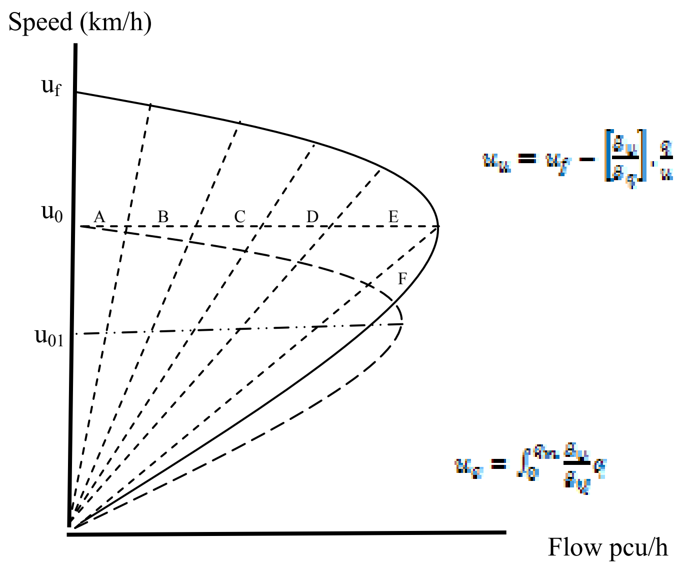



Figure 2 Hypothetical Speed-Flow Curves and LOS

Traffic volume and speed relationship is relied on in Highway Capacity Manual – HCM for the purpose of qualitative measurement of road service often termed Level of Service (LOS) in many literatures. The major problems with speed and volume estimation are two folds; firstly, it is notoriously difficult to estimate optimum speeds, attempts to simulate will often lead to exaggerated speeds with little or no resemblance to that observed; and secondly, optimum speed cannot be exceeded under uncongested traffic flow condition as often the case with LOS E. Uncongested flow terminates at optimum speed.

Although, the maximum flow at optimum speed is often taken as roadway capacity, strictly speaking its service volume aimed at measuring the quality of road service provided. Quantity as a finite element is a function of density and can only be used to measure highway capacity; beside, the equations that govern speed/flow curve do not suggest quadratic function, rather they points to negative linearity curve above and positive linearity curve below optimum speed horizon as shown in figure 2.

So, unlike the flow-density curve where congested and uncongested traffic flows operate within the concave curve finite boundaries; speed-flow curve has two distinct linear functions (negative for uncongested flow and positive for congested flow mode). So, the suggestion that LOS A to E have common optimum speed as prescribed in many literatures, distort the forceful assertion that congested and uncongested traffic flows have different optimum speeds.

## 4. APPLICATION OF PCE VALUES

The term 'passenger car equivalent' was first introduced in the 1965 Highway Capacity Manual. It was defined as 'the number of passenger cars displaced in the traffic flow by truck or a bus under the prevailing roadway, traffic and ambient conditions. This definition still holds today. Since mixed traffic stream is characterised by a variety of transport modes, vehicle volume per hour are converted into passenger car equivalent flows. PCE values for road sections with poor surfacing would obviously not be the same as those of the road sections with good surfacing, so care should be taken when applying these values.

PCE estimation methods can be summarised as; PCEs based on headways used by Cunaign (*1*), PCEs based on delay used by Cunaign (*1*), PCEs based on platoon formation used by Van Aerde and Yagar (*14*), PCEs based on speed used by Van Aerde and Yagar (*14*), PCEs based on vehicle-hours used by Sumner *et al* (*13*) and PCEs based on travel time used by Keller and Saklas (*6*). In fact Elefteriadou *et al* (*4*) in their work on development of PCE for highways suggested that of the techniques mentioned above, speed and delay were the most often used as basis for calculating PCEs on various highway types. Elefteriadou *et al* (*4*) used speed for calculating PCEs because they claim that 'speed is a performance measure immediately experienced by all uses on each type of highway, and it provides a clear picture of how smoothly a facility is operating'. This approach was suggested on the ground that speed is the principal criterion for designation of levels of service.

Van Aerde and Yagar (*14*) developed PCEs based on speed on the basis of relative rates of speed reduction related to each vehicle type. In the United Kingdom, and Malaysia, pre-determined passenger car equivalency values are usually applied to traffic volumes when converting from vehicles per hour. According to Seguin, Crowley and Zwieg *IR, (1998)* PCEs can be defined as the ratio of the mean lagging headway of a subject vehicle divided by the mean lagging headway of the basic passenger car. The headway method is one of the several techniques for measuring PCEs. By using the headway method one is implying that the relative amount of space occupied by a vehicle in motion is the basis for calculating PCE values. Headway is the distance from rear bumper of the lead vehicle to the rear bumper of the following vehicle at appoint in time. It is also a measure of separation between vehicles, which may affect safety and the ease with which pedestrians and vehicles can cross the traffic stream.

Since PCE values are central to roadway capacity calculation it follows that reduction in vehicle speeds resulting from pavement distress would also have effects on the PCE values. This was to be expected as vehicle speeds were lowered almost uniformly on road sections with poor surfacing for all types of vehicle at study sites. Observations and further analysis revealed that passenger cars often struggle to keep pace with commercial vehicles on road with poor surfacing. Traffic flows are worst when passenger cars are the platoon leaders. Therefore, the problem of passenger car equivalency values in roadway capacity analysis cannot however be ignored. On the one hand it shows the potential of commercial vehicles gaining control of roadway by exploiting the presence of poor pavement as shown in the study. On the other hand it exposed the weakness of passenger cars as mode of transport on distressed road surfaces. Given good road surface, drivers may travel at higher speeds given a certain traffic density, may keep shorter distances between vehicles ahead without lowering speed, or may choose a different lane of the carriageway. In fact drivers may even elect to change routes or departures times because of improved road and traffic conditions. For drivers who are familiar with the terrain and positioning of the defects, the travel time over the length of such road may somehow be slightly different to those who are new to or irregular users of the road.

In considering the mechanisms by which severe road surfacing condition may possibly influence roadway capacity, two groups of factors seem most important: the changing behaviour of drivers and the changing composition of traffic under prevailing conditions. Thus, when computing highway capacity, mixed vehicle volumes are usually converted into passenger car equivalency flows by taken into account vehicle type, road, traffic and ambient conditions. Ignoring PCE modifications or recalibration could lead to grossly inaccurate road capacity estimates with attendant consequences for transportation modelling.

## 6. SET UP OF FIELD STUDY

Empirical studies has shown that speed decreases exponentially with increase in density, however, it can be hypothesised that since highway capacity on road sections with poor surface conditions will be significantly different from that of the sections with good surfaces, PCE values for both sections would not be the same. In Malaysia, express, principal, state and residential roads are considered as possible study site options. Since the study relied on improvised temporary vertical deflections (ramps) across road widths, expressways and principal roads were rule out for safety reasons and regulations that ban putting ramps on such roads.

Land use, traffic and environment data were collected from sample state and residential roads. Within the objectives of the study and the site boundaries, roads were selected based on the following criteria:

- Road Geometry ≥ class 'B' road with clear visibility, level terrain and the absence of traffic signals influence
- Road Link ≥ 500m to allow for survey length > 210m, surface distress length (variable) and transition length = 160m after surface distress. The link should be free both ways of influence from road junction, roundabout, petrol station, broken down or parked vehicle, police check point and other roadway/traffic conditions that could cast doubt on data collected.
- Road must not be impaired traffic movements like on street parking, traffic signals and also sections that are free from vertical deflection not too far out of range for meaningful speed-flow measurements

The sample roads were ranked; as a result, six sites were selected knowing fully well that the study approach can be applied to all the sites with reliable outcomes. Typical survey site set up is shown below in figure 3.

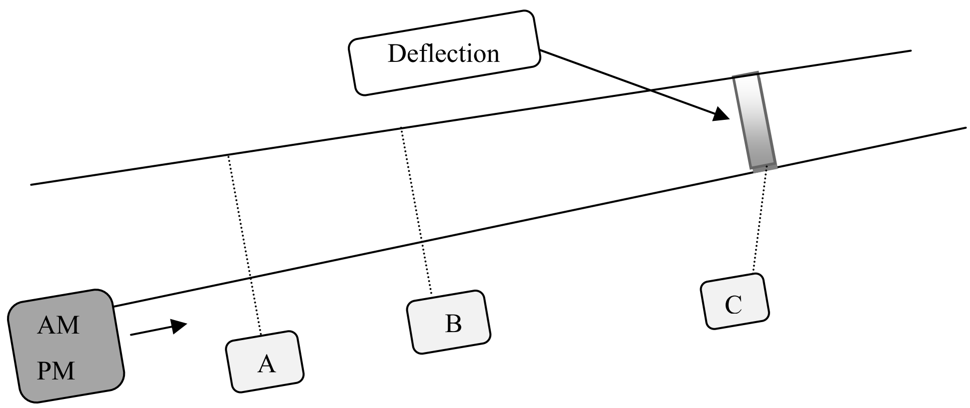



Figure 3 Typical Survey Site setup

As shown above in figure 3, study sites were divided into three sections with *Section* ***A*** as the upstream end and *Section* ***C*** the downstream end, while *Section* ***B*** was the transition part allowing for possible traffic congestion. *Section B* was set at 130m from the baseline of section A and B. Point A is spaced at 50m from B; and B is 130m from C; The spacing of road sections commenced from section C to B where the sighting distance of motorists was first estimated using observed average free flow speed of 90km/h or 55mph taken at point A. Assuming 5% gradient (G), 2.5seconds reaction time (t), 0.30 coefficient of friction (f), and stopping distance, $S = 0.278Vt + 0.039\ V^{2} / a \approx 130\text{m}$ Where: V = design speed, mph (km/h), t = brake reaction time, 2.5 s, a = driver deceleration, $\text{m/s}^2$.

The upstream end (section A) is the section with good surface while the downstream end (section C) is the section with vertical deflection in form of edge to edge round top humps 100mm high in the direction of traffic. Volume and speed and vehicle type data were collected using automatic traffic counter for 30 days. Data on the peak and off peak periods collected at the initial stage were used to establish the observation time period in order to eliminate the effect of peak period on capacity because the primary focus is an uninterrupted flow. Data relating to rainfall were used to exclude rainy periods. Thus the influence of rain on capacity was eliminated and surveys were conducted during daylight to eliminate the effect of darkness.

Initial test runs and analysis were carried out randomly at location J008; influence of round top humps on speed were tested at 75mm, 100mm and 150mm in order to establish an appropriate study height, experienced gained proved invaluable when full study was conducted.

## 7. FIELD STUDY RESULTS AND ANALYSIS

The main aim to determine capacity and level of service for 6 surveyed sites based on equation 2 shown below. The objectives are as follows: Determine traffic flow from vehicle volume and density from the speed / flow relationship relying on the fundamental diagram then use the flow and density relationship to determine the capacities for road sections A and B; extrapolate optimum speeds from the computed capacities to determine level of service.

$$q = -\beta_o + \beta_1 k - \beta_2 k^2 \qquad (2)$$

By computing roadway capacity for each link section, it is recognised that capacity varies per road section and the method used for estimating capacities is based on the fundamental relationship between flow, speed and density. In the flow (*q*) – density (*k*) relationship density is used as the control parameter and flow is the objective function.

However, before roadway capacity can be estimated it is important that the effect of mixed traffic is taken into consideration and this was done by way of volume conversion into passenger car equivalency units. PCE is usually the terminology employed in the United States and Canada, while PCU is commonly used in the United Kingdom. PCE and PCU are same. Compare the capacities in section A and B to establish whether a loss resulting from modified PCE values has occurred.

The calculation of capacity by way of quadratic function is incomplete without determining the point of the extrapolated curve that represents the capacity and this point is a function of critical density. This critical density can be derived, estimated or assumed as appropriate. It is quite possible to extrapolate mathematically until the maximum of the q-k function is attained, but how will such theoretical values compare with reality of traffic operations? The calculated capacities may be unrealistically high and questionable. It can be argued that capacities so derived may very little resemblance to traffic conditions.

In any case empirical density is often difficult to determine because one should observe a complete and uniform road section and count the total number of cars present at any moment. Instead, local density is used in the calculations. An assumed critical density can be used to calculate the corresponding rate of flow. But it has to be known. So it is now obvious that an assumed critical density value and mathematical model of critical density cannot be made in this case in the light of the stated poor surface conditions, hence the need to estimate critical density. In the analysis only data per carriageway lane were used, therefore the estimated capacities are carriageway lane capacities. Tables 1 and 2 contain some of the variables used for capacity analysis.

The effects of pavement distress on passenger car equivalent values are significant and must be taken into account when determining flow at road section with pavement distress. From observation at surveyed sites, trucks are less affected by pavement distress than passenger car and it may be argued that the passenger car equivalent values of trucks or HGVs are somewhat lower than those of passenger cars on roadways with significant pavement distress.

It can be mentioned in passing that there was a sharp difference in the attitude of heavy goods vehicle (HGV) drivers on road section with surfacing distress. HGV motorists pay very little attention to pavement distress as observed at surveyed sites. It may be argued that because of change in drivers attitude relative to pavement distress, and to some extent the need by heavy goods vehicle (HGV) operators to make profit, a description that depict passenger car equivalency value for HGV as substantially higher than one unit on distressed level terrain is somehow distorted. Also, the extent to which HGV vehicles are driven at an above average speed on such terrain would give to claim that HGV passenger car unit is about one unit sometimes lower than one unit.

The study employed a simplistic approach based on Greenshields (*1934*) headway method of calculating PCE between vehicles under saturated flow conditions as:

$$PCE_{ij} = H_{ij} / H_{pcj}$$

Where, *$PCE_{ij}$ is the PCE of vehicle Type i under Conditions j, and $H_{ij}$, $H_{pcj}$ is the average headway for vehicle Type i and passenger car for Conditions j.* The results of the computed PCE values are shown below in Tables 1 & 2. By using the headway method one is implying that the relative amount of space occupied by a vehicle in motion is the basis for calculating PCE values. Same PCE estimation method was applied to the road section with vertical deflection for the purpose of consistency in application.

From observations at survey sites, passenger cars sometimes force HGVs to slow down especially when they are platoon leaders because of their manoeuvrability difficulties on road sections with vertical deflection. These observations further validate the definition of PCE values and to some extent the reason why the PCE values of HGVs and LGVs could be slightly less than 1.0 given unfavourable conditions. It is worth noting that PCE values are dynamic and tasked with presenting the effect of mixed traffic stream for capacity and level of service analysis.

**Table 1** Estimated PCE values for Good Road Section

| Site | Vehicle Type | Speed m/sec | Density Veh/hr | Spacing M/veh | Headway sec/veh | PCE unit |
|---|---|---|---|---|---|---|
| 004 | PC | 22 | 28 | 35.714 | 1.623 | 1.000 |
| | LGV | 17 | 28 | 35.714 | 2.101 | 1.294 |
| | HGV | 11 | 28 | 35.714 | 3.247 | 2.000 |
| 005 | PC | 22 | 22 | 45.455 | 2.066 | 1.000 |
| | LGV | 18 | 22 | 45.455 | 2.525 | 1.222 |
| | HGV | 13 | 22 | 45.455 | 3.497 | 1.692 |
| 006 | PC | 22 | 31 | 32.258 | 1.466 | 1.000 |
| | LGV | 17 | 31 | 32.258 | 1.898 | 1.294 |
| | HGV | 15 | 31 | 32.258 | 2.151 | 1.467 |
| 008 | PC | 22 | 34 | 29.412 | 1.337 | 1.000 |
| | LGV | 19 | 34 | 29.412 | 1.548 | 1.158 |
| | HGV | 15 | 34 | 29.412 | 1.961 | 1.467 |
| 009 | PC | 24 | 32 | 31.250 | 1.302 | 1.000 |
| | LGV | 16 | 32 | 31.250 | 1.953 | 1.500 |
| | HGV | 14 | 32 | 31.250 | 2.232 | 1.714 |
| 011 | PC | 24 | 26 | 38.462 | 1.603 | 1.000 |
| | LGV | 17 | 26 | 38.462 | 2.262 | 1.411 |
| | HGV | 14 | 26 | 38.462 | 2.747 | 1.714 |
| PCE | PC = 1.00 | | LGV 1.37±0.04 | | HGV = 1.75±0.07 | |

**Table 2** Estimated PCE values for Poor Road Section

| Site | Vehicle Type | Speed m/sec | Density veh/hr | Spacing m/veh | Headway sec/veh | PCE Unit |
|---|---|---|---|---|---|---|
| 004 | PC | 10.56 | 50 | 20.000 | 1.895 | 1.000 |
| | LGV | 11.11 | 50 | 20.000 | 1.800 | 0.950 |
| | HGV | 10.83 | 50 | 20.000 | 1.846 | 0.974 |
| 005 | PC | 10.83 | 31 | 32.258 | 2.978 | 1.000 |
| | LGV | 11.11 | 31 | 32.258 | 2.903 | 0.975 |
| | HGV | 10.56 | 31 | 32.258 | 3.056 | 1.026 |
| 006 | PC | 10.28 | 56 | 17.857 | 1.737 | 1.000 |
| | LGV | 10.83 | 56 | 17.857 | 1.648 | 0.949 |
| | HGV | 10.28 | 56 | 17.857 | 1.737 | 1.000 |
| 008 | PC | 10.83 | 48 | 20.833 | 1.923 | 1.000 |
| | LGV | 11.94 | 48 | 20.833 | 1.744 | 0.907 |
| | HGV | 11.11 | 48 | 20.833 | 1.875 | 0.975 |
| 009 | PC | 09.72 | 51 | 19.608 | 2.017 | 1.000 |
| | LGV | 10.56 | 51 | 19.608 | 1.858 | 0.921 |
| | HGV | 10.00 | 51 | 19.608 | 1.961 | 0.972 |
| 011 | PC | 10.56 | 51 | 19.608 | 1.858 | 1.000 |
| | LGV | 10.83 | 51 | 19.608 | 1.810 | 0.974 |
| | HGV | 10.56 | 51 | 19.608 | 1.858 | 1.000 |
| PCE | PC = 1.00 | | LGV=0.95±0.01 | | HGV=0.98±0.01 | |

After computing PCE values based on survey data, they were used to estimate highway capacities for road section with and without good surface. Equation 2 was relied on for quantitative measurements as shown below;

$$q = -\beta_o + \beta_1 k - \beta_2 k^2 \qquad (2)$$

By differentiating flow wrt density equation 2 can be rewritten as:

$$\partial q / \partial k = \beta_1 - 2\beta_2 k$$

By setting $\partial q / \partial k = 0$;

Determine critical density ($k_c$), and then plug $k_c$ into equation 2 to determine maximum flow ($q_m$)

$$q_m = -\beta_o + \beta_1 k_c - \beta_2 k_c^2$$

Test model equations for statistical fit at 5% level of significance

Determine the optimum speeds

For $q = uk$;

Optimum Speed, $u_0 = q_m / k_c$

### 7.1 Sample calculations for site J008

Road without vertical deflection section A

$$q_A = -1.5755k^2 + 105.47k - 59.044$$

$$\partial q / \partial \kappa = 2(-1.5755k) + 105.47 = 0$$

$$\kappa_{crt} = 34 \text{ veh/km}$$

$$q_A = -1.5755(34)^2 + 105.47(34) - 59.044$$

$$q_A = 1706 \text{ pcu/hr}$$

$$u_o = 1706/34 \approx 50 \text{ km/h}$$

Road with vertical deflection section B:

$$q_B = -0.5022k^2 + 47.701k - 10.25$$

$$\partial q / \partial \kappa = 2(-0.5022k) + 47.701 = 0$$

$$\kappa_{crt} = 48 \text{ veh/km}$$

$$q_B = -0.5022(48)^2 + 47.701(48) - 10.25$$

$$q_B = 1123 \text{ pcu/hr}$$

$$u_o = 1123 / 48 \approx 23 \text{ km/h}$$

Optimum speeds were extrapolated from the computed capacities. By computing roadway capacity for each link section, it is recognised that capacity varies per road section and the method used for estimating capacities is based on the fundamental relationship between flow, speed and density. In the flow (*q*) – density (*k*) relationship, density is used as the control parameter and flow is the objective function. Summary of the model coefficients for all surveyed sites are shown below in Table 3.

**Table 3** Summary of Model Coefficients

| Site | | Density $-\beta_o$ | Speed $\beta_1 k$ | Flow $-\beta_1 k^2$ | $R^2$ |
|---|---|---|---|---|---|
| J001 | Without VD | 158.26 | 103.22 | 2.0398 | 0.87 |
| | With VD | 43.569 | 51.853 | 0.6396 | 0.96 |
| J002 | Without VD | 14.001 | 103.22 | 2.4680 | 0.95 |
| | With VD | 124.48 | 61.456 | 0.9628 | 0.98 |
| J003 | Without VD | 91.271 | 113.61 | 2.0224 | 0.97 |
| | With VD | 140.11 | 58.934 | 0.6886 | 0.97 |
| J004 | Without VD | 56.201 | 106.22 | 1.7856 | 0.96 |
| | With VD | 4.5358 | 46.901 | 0.6113 | 0.95 |
| J005 | Without VD | 52.064 | 111.86 | 1.9201 | 0.96 |
| | With VD | 107.66 | 57.765 | 0.8927 | 0.96 |
| J006 | Without VD | 102.12 | 128.85 | 2.6756 | 0.98 |
| | With VD | 73.528 | 52.704 | 0.6681 | 0.98 |

Test statistics for model equations in the study are found to be reliable at 5% level of significance, all r are greater than 0.5, ts' greater than 2.5 and Fs' greater than 10.0 Computed highway capacities and LOS analysis for all sites are summarised below in tables 4 and 5.

**Table 4** Summaries of Highway Capacity & LOS Analysis – Section A

| Site | Static PCE Values | | | | Dynamic PCE Values | | | |
|---|---|---|---|---|---|---|---|---|
| | Highway Capacity pcu/h | Critical density veh/km | Optimum speed km/h | LOS | Highway Capacity pcu/h | Critical density veh/km | Optimum speed km/h | LOS |
| J001 | 1555 | 28 | 56 | B | 1650 | 29 | 56 | B |
| J002 | 1111 | 22 | 51 | B | 1065 | 21 | 51 | B |
| J003 | 1663 | 31 | 54 | B | 1504 | 28 | 54 | B |
| J004 | 1706 | 34 | 50 | B | 1523 | 30 | 51 | B |
| J005 | 1713 | 32 | 54 | B | 1577 | 29 | 54 | B |
| J006 | 1560 | 26 | 60 | A | 1449 | 24 | 61 | A |

**Table 5** Summaries of Highway Capacity & LOS Analysis – Section B

| Site | Static PCE Values | | | | Dynamic PCE Values | | | |
|---|---|---|---|---|---|---|---|---|
| | Highway Capacity pcu/h | Critical density veh/km | Optimum speed km/h | LOS | Highway Capacity pcu/h | Critical density veh/km | Optimum speed km/h | LOS |
| J001 | 1204 | 50 | 24 | E | 1007 | 41 | 25 | E |
| J002 | 849 | 31 | 27 | D | 856 | 32 | 27 | D |
| J003 | 1468 | 56 | 26 | D | 1121 | 43 | 26 | D |
| J004 | 1122 | 48 | 23 | E | 895 | 38 | 23 | E |
| J005 | 1195 | 51 | 23 | E | 827 | 32 | 26 | D |
| J006 | 1244 | 51 | 24 | E | 966 | 39 | 24 | E |

From tables 4 and 5 above, it can be seen that capacities at section A are substantially higher than those at section B for all investigated sites. Also critical densities at road section B were significantly higher than those at section A. Vehicles operating at road section A were completely unaffected by the road hump. They were almost completely unimpeded in their ability to manoeuvre within the traffic stream because the operating conditions afford the driver higher speeds. Whereas at road section B drivers were operating at lower speeds because freedom to manoeuvre within the traffic stream was limited due to poor surfacing.

Because critical densities have been found to be higher at the road section B; it follows that spacing would be smaller. Speeds on this road section for all types of vehicles are almost the same because all the vehicles were constrained by the same road conditions with very little room for manoeuvrability. Thus, given smaller spacing, lower speed, and larger density, it's to be expected that the PCE values for road sections with vertical deflection would be somewhat lower than that of the section without vertical deflection.

The estimated PCE values for road section A, were found to be lower than those presently used for most of the standard capacity analysis (PC=1, LGV=1.5 HGV=2.1) and this may not be unconnected with the low level of traffic volume. This may call to question the appropriateness of the static Malaysia PCE values. However, the derived PCE values are preliminary findings and separate large scale studies are needed in order to derive a true reflective PCE values.

Notwithstanding, investigation into the application of passenger car equivalency values in highway capacity and level of service analysis has revealed that: i) poor road surfacing has significant effects on PCE values; ii), given poor road condition, PCE values for HGV are not substantially higher than 1; iii) That the estimated highway capacity are affected PCE values; iv) LOS is unaffected by PCE values.

In the Highway Capacity Manual-HCM, level-of-service (LOS) is taken as a measure-of-effectiveness. It uses the letters A through F to describe prevailing road traffic condition, with A being best and F being worst., The suggestion that LOS A to E have optimum speed commonality as shown in many literatures distorts assertion that congested and uncongested capacities have different optimum speeds as shown in the paper. According to HCM 2000 LOS E is a marginal service state. Flow becomes irregular and speed varies rapidly, but rarely reaches the posted limit. On highways this is consistent with a road operating over its designed capacity.' Is HCM 2000 suggesting that LOS is based on design capacity that can be exceeded by absolute capacity?

Clearly, under LOS E condition, the roadway is at capacity. However, the problem with LOS E is that absolute capacity is exceeded because the bottom range of LOS E is below optimum speed ($u_0$), when it should have terminated at $u_0$. LOS E description distorts negative linearity rule of lower boundary termination at maximum flow. HCM 2000 further introduced negative diagonal speed-flow curves suggesting that corresponding concave capacity shifts will occur along several lines with the same optimum speed. Again this is doubtful.

Further, HCM 2000 suggested that 'LOS F is the lowest measurement of efficiency for a road's performance. Flow is forced; every vehicle moves in lockstep with the vehicle in front of it, with frequent drops in speed to nearly zero kph'. Technically, a road in a constant traffic jam would be at LOS F, because LOS does not describe an instant state, but rather an average or typical service'. In sum, highway traffic flow can be categorized into two modes, uncongested and congested. Uncongested traffic mode depicts a highway where traffic demand is less than the absolute capacity, vehicle speeds oscillating between optimum and free flow speed limits under prevailing conditions. Whereas congested traffic mode depicts a highway where capacity is over sub-subscribed, speeds become unpredictable, quality of service significantly reduced, vehicles are herd and synchronised.

From the discussion it can be seen that roadway capacity analysis and their attendant PCE values encapsulate effects resulting from three variables, namely density, speed and flow under prevailing conditions. Even though attempts have been made by many scholars to address the issues of passenger car equivalency and provide more realistic values for uninterrupted flow, there are no uniform values. So, by using a simplistic 'headway of vehicle type' approach we can at least point PCE values in a particular direction.

## 7 CONCLUSIONS

Although several capacity-estimation methods are based on appropriate theories concerning macroscopic traffic flow, the attempts to determine the capacity of a road by existing methods will generally result in a capacity value estimate. The validity of this value is hard to investigate because of the lack of a reference capacity value, which is supposed to be absolutely valid. Nevertheless the chosen method in the paper would be sufficient in determining capacity in case where capacity is seldom reached. The paper has shown that road conditions among others have implications for PCE values, so it can be concluded that:

- There is a significant change in roadway capacity resulting from PCE adjustments.
- Capacity is generally higher with standard PCE values compared to adjusted PCE values.
- There is no significant change in level of service due to adjusted PCE values.
- There is no other factor other than vertical deflection that affected capacity and LOS changes.
- The hypothesis that vertical deflection has influence on PCE values remains valid.
- The hypothesis that capacity and LOS loss would result from vertical deflection remain valid.

The conclusions drawn follow from the application of PCE values in highway capacity and level of service analysis. Further research is needed to ascertain the reliability of PCE values under prevailing conditions. In this study headway was estimated from spacing and speed, it would be useful to conduct a larger headway distribution survey for vehicle types under varying conditions.

## REFERENCES


1. Cunagin, W.D., and C.J. Messer. 1982 *Passenger Car Equivalents for Rural Highways.* Report FHWA/RD-82/132, FHWA, U.S. Department of Transportation, Washington D.C
2. Elefteriadou, L *et al* (*1998*) *Development of Passenger car Equivalents for Freeways, Two-Lane Highways and Arterials*, Pennsylvania transportation Institute 201 Research Office Building, University Park PA
3. Greenshields. B.D,. 1934 *A Study of Traffic Capacity,* Proceedings of the Highway Research Board. HRB 1934) Vol. 14, pp 448-477
4. *Highway Capacity Manual (1985)*, Special Report 209, Transportation Research Board (TRB), National Research Council, Washington, D.C
5. *Highway Capacity Manual 2000* Transportation Research Board (TRB), National Research Council, TRB, National Research Council, Washington, DC
6. Keller, E.L and Saklas, J.G *1984 Passenger car Equivalents From Network Simulation.* Journal of Transportation Engineering, Vol 110 No 4 July 1984 pp 397-411
7. Minderhoud, M H.Botma and PH Bovy 1997 *Assessment of Roadway Capacity Estimation Methods*. In Transportation Research Record 1572 TRB National Research Council Washington DC (pp 59-67)

8. Papendrect, H. Bothma, H. and D. Westland, 1980 *Validation of Capacity Estimators Based on the Decomposition of the Distribution of Headways.* Delft University of Technology, Delft.
9. Persuad, B., and V.Hurdle. 1991 Freeway capacity: definition and measurement issues. In *Highway Capacity and Level of Service* (Brannolte, ed), Balkema, Rotterdam, , pp 289-307.
10. Skabadornis. A *et al* 1996 *The 1-880 Field Experiment: Database Development and Incident Delay Estimation procedures.* In transportation research Record 1554, TRB National Research council, Washington D.C
11. TRB (Transportation Research Board), 1994 Special Report 209: "Highway Capacity Manual", third ed. Transportation Research Board (TRB), Washington, DC.
12. Seguin, E.L. Crowley, K.W., and Zweig, W.D., 1998, *Passenger Car Equivalents on Urban Freeways.* Interim Report, contract DTFH61-C00100, Institute for Research (IR), State College, Pennsylvania.
13. Sumner, R.Hill. D., and Shapiro, S. Segment Passenger Car Equivalent Values for Cost Allocation on Urban Arterial Roads, Transportation Research, Vol.18A No 5/6 Dec 1984 pp 399-406.
14. Van Aerde, M and S.Yagar. 1984 *Single Regime Speed – Flow- Density Relationship for Freeways and Arterials.,* Presented at the 74th Annual Meeting, Transportation research TRB, National Research Council, Washington, D.C.,

# THE EFFECTIVENESS OF CONGESTION COUNTERMEASURES BY USING LED IMFORMATION BOARDS AT CHUO EXPRESSWAY IN JAPAN

**Masahiko MITSUHASHI**
*Traffic Engineering Team, Hachioji Branch, Central Nippon Expressway Co., Ltd.*
*231 Utsuki-cho, Hachioji, Tokyo, Japan 192-8648*
*m.mitsuhashi.aa@c-nexco.co.jp*
**Toshiyuki MORI**
*International Team, Road-Related Business Development Bureau, Head Office,*
*Central Nippon Expressway Co., Ltd.*
*Mitsui-Sumitomo Nagoya Bldg., 10F, 2-18-19, Nishiki, Naka-ku, Nagoya, Japan 460-0003*
*t.mori.ad@c-nexco.co.jp*
**Soichi MINAKAWA**
*Hachioji Maintenance/Customer Service Center, Hachioji Branch,*
*Central Nippon Expressway Co., Ltd.*
*287-1 Utsuki-cho, Hachioji, Tokyo, Japan 192-0024*
*s.minagawa.aa@c-nexco.co.jp*

**ABSTRACT**

This paper describes the effectiveness of countermeasures by using Light Emitting Diode (LED) information boards, which is one of the countermeasures recently attempted to reduce congestion on Chuo Expressway, Japan. The LED information boards, installed near the bottlenecks at sag and merging sections, provide drivers with information that is enable to urge the drivers to accelerate or to adjust the balance of traffic lane utilization. In order to evaluate the performance with LED information boards, an experiment was executed in a study section where congestion often occurs and a comparison analysis of flow rate, speed and lane utilization rate between with and without LED information boards was undertaken. The results showed that the present LED information boards have some effects on improving flow rate, speed and lane utilization rate at bottlenecks. We concluded that the countermeasures using LED information boards would be one of the effective countermeasures in reducing congestion at low cost without changing road structure.

**1. INTRODUCTION**

Traffic Congestion seriously affects our economy, quality of life and environment not only in Japan but also in any countries all over the world. Therefore, mitigating congestion is a high priority for us. Chuo Expressway, from Tokyo to Nagoya, is one of the most important components of the transportation network in Japan. On the Expressway in large cities and vicinities, congestion is often caused by weekend recreational traffic or workday. Much of the congestion is due to geometric bottlenecks such as sag sections, merging areas and tunnels. In order to alleviate congestion, it is clear that adding more lanes to existing expressways is very effective. In some metropolitan area, however, it is difficult to undertake major road expansions mainly because of construction costs and right-of-way constraints. Since road building alone will not solve the problem, a variety of countermeasures have been carried out to make best use of existing expressways, such as implementing traffic demand management for weekend recreational traffic, providing real-time information on present traffic conditions and traveler information of congestion forecast especially for holiday season when traffic demand increases.
As well as the above mentioned countermeasures to make best use of existing expressways, the countermeasures using LED information boards is used on Japan's Expressways. The present

countermeasures have already been implemented in several sections on Japan's Expressways. To examine the case of countermeasures using LED information boards, latest information was collected from some literatures (from [1]-[4]), internet website (from [5], [6]) and some transportation practitioners. The collected information is described in the following chapter.

## 2. EALIER EXAMPLES

The countermeasures using LED Information Board have already been executed in several sections on Japan's Expressways. From ealier examples we collected, it is found that the intended use of LED information boards is made to classify two types depending on causes of congestion. One is a type to provide speed recovery information to drivers near the bottleneck in sag sections. Sag sections are known as bottlenecks as well as some obvious bottlenecks such as merging or diverging sections and weaving sections on Japanese Expressways. From many earlier empirical studies, its cause showed that drivers frequently slow down unconsciously in sag sections because they are not aware of grade change. Traffic also concentrates in inner lanes where congestion begins with available traffic capacity not fully utilized. The other is a type to be installed in the vicinity of merging area, and to provide information that improve lane utilization rate. It is fact that traffic congestion frequently occurs at merging area, especially during heavy traffic demand, because entering and exiting traffic causes disturbances to traffic on multilane facilities.

### 2.1 An Example Of Measures At Sag Section

Figure 1 shows a case where measures are executed in the consecutive sag sections on Tomei Expressway. The present LED information boards provide drivers with information that is enable to urge the drivers not to fall speed down in the vicinity of each bottleneck before congestion occurs in order to delay occurrence of congestion. Once congestion occurs, the displayed message of the boards changes the speed recovery information. Messages displayed on the boards change automatically in response to traffic conditions with the aim of improving traffic flows. According to asking to the on-site transportation practitioners, it appears that the execution of measures is effective in alleviating congestion, compared without measures.

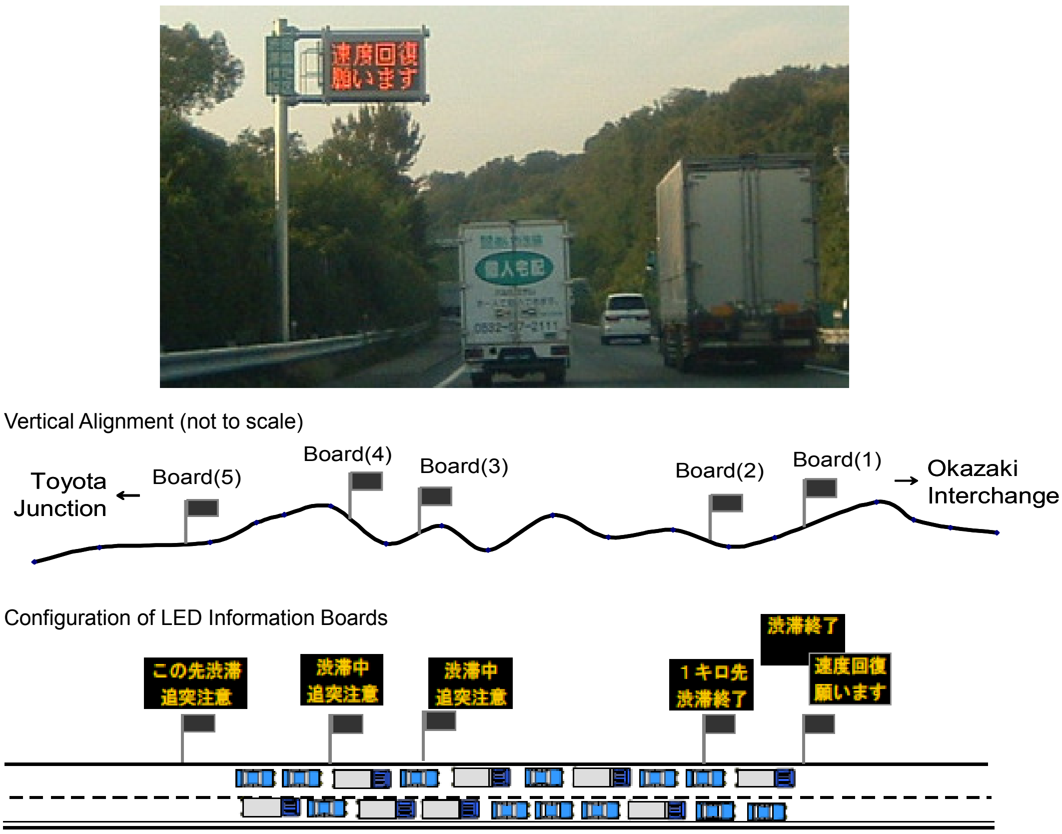



Figure 1: LED Information boards at sag section
(Okazaki Area on Tomei Expressway)

### 2.2 An Example Of Measures At Merging Area

The following case is the second type of measures using LED information boards as shown in Figure 2. In order to prevent congestion from occurring during high traffic volumes caused by inequal lane utilization of two lanes at merging area, LED sign cars is installed at the downstream of merging area. Entering drivers from on-ramp are provided with “Keep Left” message immediately after merging and can choke off transferring from outer lane to inner lane. The present measure reports positive results that LED sign cars improve traffic conditions on speed-flow relationship and lane utilization rate (from [2]).

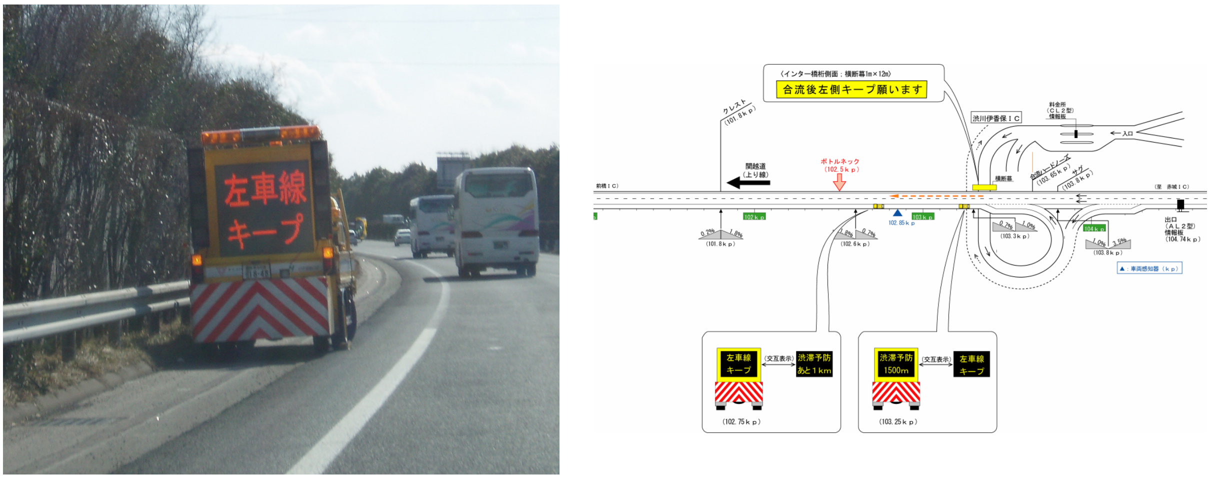



Figure 2: LED sign cars installed on the hard shoulder
(Shibukawa-Ikaho Interchange on Kanetsu Expressway)

## 3. STUDY OBJECTIVES

Besides the above-mentioned, the countermeasures using LED information boards or LED sign cars have been implemented at many sites on Japan’s Expressways and showed good results in reducing congestion. However until now, there is no case of having carried out simultaneously the present two types of measures in the consecutive section including two bottlenecks where merging and sag sections are adjacent. We selected as study section such a consecutive section including two bottlenecks on Chuo Expressway. The objective of the empirical study is to perform the experimental measures with LED information boards, and to evaluate the performance on traffic conditions, that is, flow rate, speed and lane utilization rate.

## 4. STUDY AREA AND METHOD OF EXPERIMENTAL COUNTERMEASURE

The study area is an inbound two-lane section of Chuo Expressway. This section is located about 20 km west from the center of Tokyo, where the congestion occurs over 150 times a year because of carrying traffic to the central business district of Tokyo during the morning peak period. Some earlier researches report that this section has two bottlenecks. One is merging section located around 6.8 kilometer-post(kp), which is called “Chofu Interchange Merging Area”, and the other is sag section located around 5.0 kp, which is called “Jindaiji Bus Stop Sag”. Figure 3 shows vertical alignment, plan view, location of the above two bottlenecks, and configuration of LED information boards in the study area.

### 4.1 Installation of the LED Information Board

As shown in Figure 4, the LED information boards were installed temporarily on the hard shoulder in the study area, which is equal to the type of LED information boards often used on worksite. In general, LED sign cars is used when experimented on other Japan’s Expressways, but the LED sign cars cannot be put in the study section because the one is bridge sections with narrow hard shoulder. Therefore, we selected the temporary LED information boards in this study section, and also the experimental period was limited at the peak time in the morning when congestion often occurs.

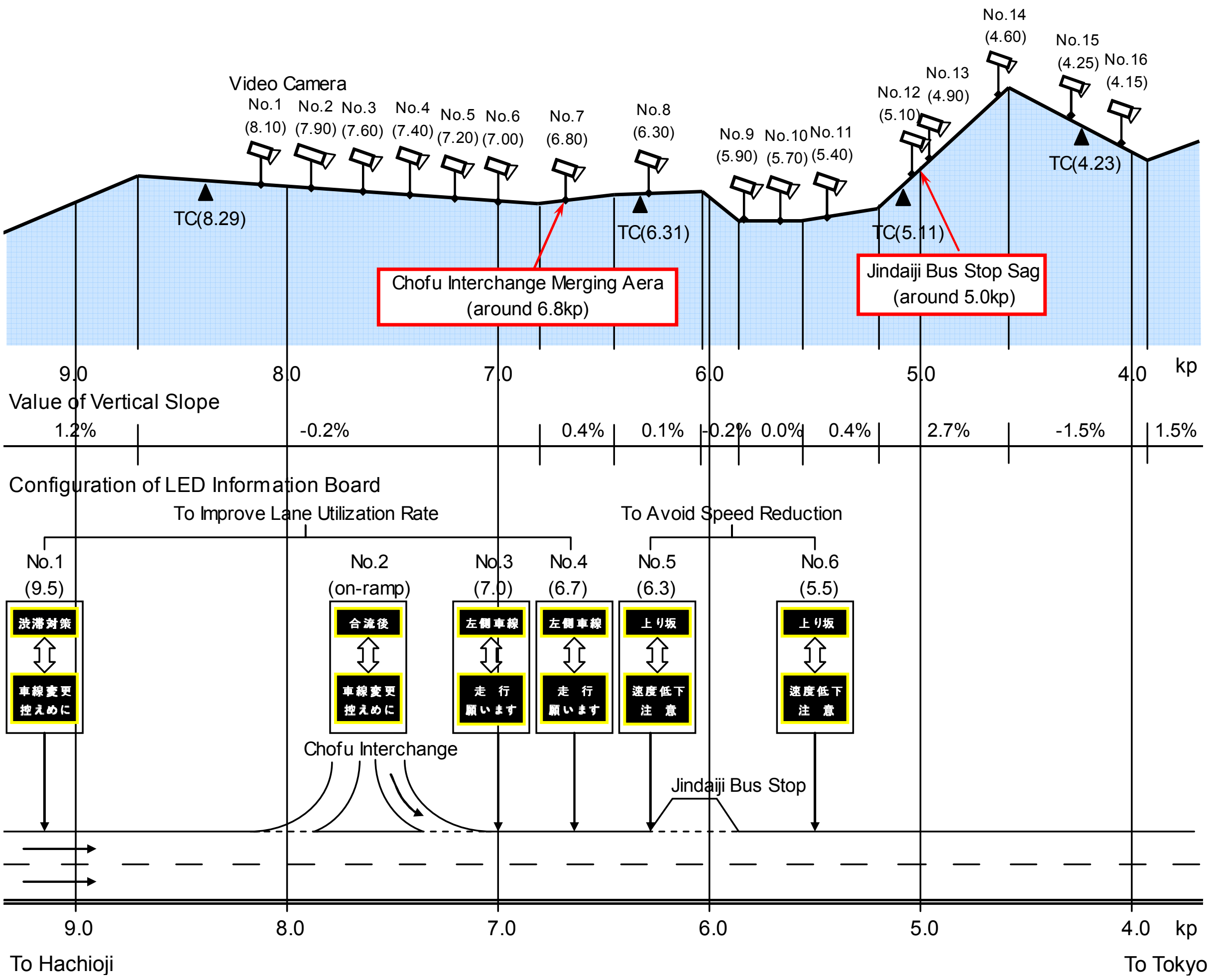



Figure 3: Vertical alignment, plan view, configuration of LED information boards


Figure 4: LED information board set up temporarily on the hard shoulder
(In the vicinity of Chofu Interchange on Chuo Expressway)

### 4.2 Display of the LED Information Board

Messages displayed on the LED information boards are shown in Table 1. As for the LED information boards from No.1 to No.4 installed in merging area, the messages on the boards to improve the lane utilization rate is offered to drivers. At the same time, the messages from No.5 to No.6 installed in sag sections provide drivers with information in order to make them recognize that there is an upgrade section ahead reducing speed easily.

Table 1: Display of the LED information board

| LED Information Board No. | Display Message | Installation Position | Main Purpose of Measures |
|---|---|---|---|
| No.1 | Congestion Measures<br><-----><br>Hold back Lane Change | 9.5kp | To improve lane utilization rate |
| No.2 | After Merging<br><-----><br>Hold back Lane Change | Chofu Interchange on-ramp | |
| No.3 | Left Lane<br><-----><br>Go Ahead | 7.0kp | |
| No.4 | Left Lane<br><-----><br>Go Ahead | 6.7kp | |
| No.5 | Upgrade Ahead<br><-----><br>Caution! Speed Down | 6.3kp | To avoid speed reduction |
| No.6 | Upgrade Ahead<br><-----><br>Caution! Speed Down | 5.5kp | |

<----->: Alternate message

## 5. PERFORMANCE MEASURE EVALUATION

### 5.1 Data Collection

In the study section, congestion often occurs in the morning on weekdays. Therefore, the experiment was performed for 8 weekdays in December of 2008 as shown in Table 2. The experimental period was from 6:00 a.m. to the end of congestion each day. The data used for the comparison analysis with and without LED information boards was obtained from detectors which include volume, average spot speed, heavy vehicle composition and occupancy. In order to get data about traffic volume, speed, and lane changing maneuvers especially at merging sections, video cameras were also used at all sixteen locations. The cameras faced in the same direction as traffic. The camera data were collected from December 15 to December 19. The location of detectors and video cameras is shown in Figure 3.

### 5.2 Method of Analysis

In the study, two types of experimental measures using LED information boards were carried out. One is the measures against “Chofu Interchange Merging Area” where LED information boards provide drivers with information to improve the lane utilization rates of both lanes from the upstream and the downstream in merging sections around Chofu Interchange. The other is the measures against “Jindaiji Bus Stop Sag” where providing drivers with information to urge the drivers to avoid speed reduction or to accelerate.

In the analysis, the above two types of measures were evaluated respectively because the station where the data used for analysis was obtained from the detectors and the video cameras was different.

When evaluating the performance with LED information boards, indicators to be addressed by the evaluation are three kinds of data, that is, flow rate, speed and lane utilization rate between with and without LED information boards.

Figure 6 shows speed contour maps during the experiment period where data is obtained by detectors installed along expressway with 5-minute mean speed. As can been seen in Figure 6, congestions occurring in the downstream reached the experiment section depending on the traffic conditions each day. Such data were excluded in the comparison analysis.

Table 2: Date of the experiment with or without LED information board

| Date | With or Without LED Information Board | Bottlenecks | Period of Analysis | Time number of Analysis |
|---|---|---|---|---|
| Dec. 15 Mon | Without | Chofu Interchange Merging Area | 6:00-8:45 | 2:45 |
| Dec. 16 Tue | With | Chofu Interchange Merging Area | 6:00-7:40 | 1:40 |
| Dec. 17 Wed | Without | Jindaiji Bus Stop Sag | 6:00-10:15 | 4:15 |
| Dec. 18 Thu | With | Chofu Interchange Merging Area | 6:00-8:20 | 2:20 |
| Dec. 19 Fri | Without | Jindaiji Bus Stop Sag | 6:00-9:35 | 3:35 |
| Dec. 22 Mon | With | Jindaiji Bus Stop Sag | 6:00-9:35 | 3:35 |
| Dec. 25 Thu | With | Jindaiji Bus Stop Sag | 6:00-9:40 | 3:40 |
| Dec. 26 Fri | With | Jindaiji Bus Stop Sag | 6:00-9:10 | 3:10 |

### 5.3 Flow Rate Before and After Congestion With / Without LED Information Board

In the analysis, two types of flow rate are compared with and without LED information boards. One is breakdown flow rate (or capacity just before congestion) and the other is queue discharge flow rate (or capacity during congestion). Occurrence of congestion or breakdown is assumed for a traffic condition when 5-min space mean speed drops below 40km/hr and it continues for over 15 minutes. Breakdown flow rate is defined as an averaged value in 5-minite of vehicles observed in a 15-minute just before congestion, while queue discharge flow rate is defined as an average flow rate during congestion. An example of two types of flow rate are shown in Figure 5.

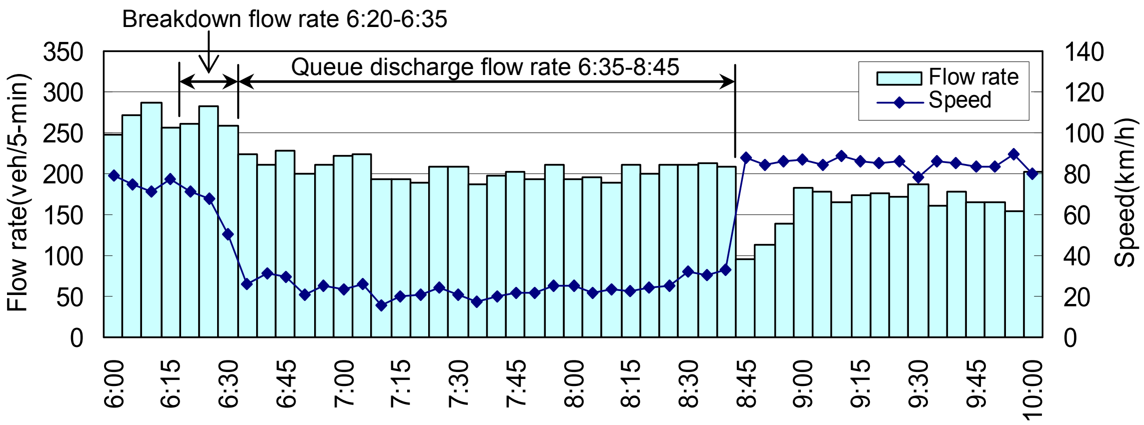



Figure 5: Definition of breakdown flow rate and queue discharge flow rate
(8.1kp Inbound of Chou Expressway, Dec. 15, 2008)

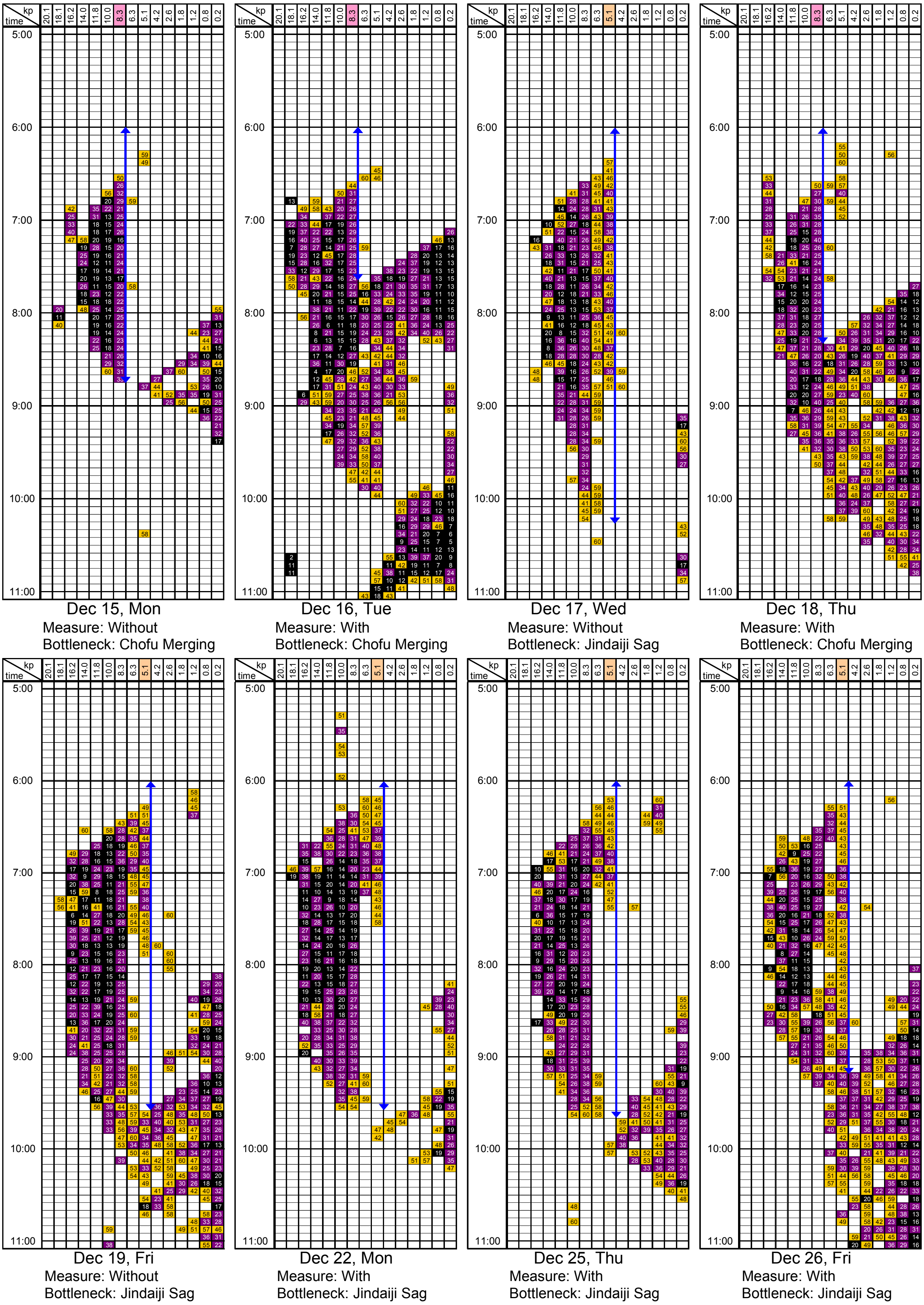



Figure 6: Speed contour map for the experiment period

(LEGEND) 5-min space mean speed in all lanes:

15 ≦ 20 km/h   20 km/h < 25 ≦ 40 km/h   40 km/h < 50 ≦ 60 km/h   60 km/h <

Period of analysis: ↔

Based on the above, the results on two types of flow rates are presented as shown in Figure 7 and 8. Compared with the case without LED information boards at “Chofu Interchange Merging Area” by the day (Figure 7), the above both flow rates with LED information boards increases. Likewise compared, at “Jindaiji Bus Stop Sag” (Figure 8), it was only a day (Dec 22) that some increase of breakdown flow rates with LED information boards was observed. Queue discharge flow rates with LED information boards are also stayed in a slight increase. In this, it seems that it is not property evaluated due to the lack of the number of sample data defined above (refer to Figure 6).

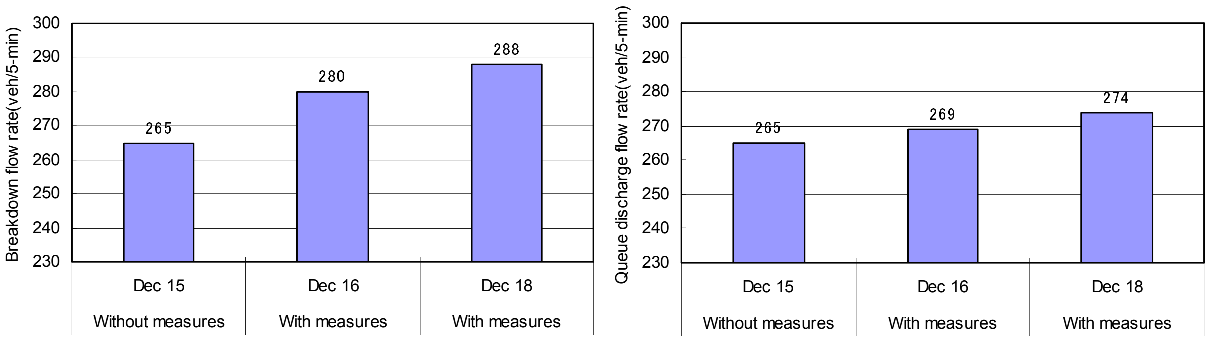



Figure 7: Changes in flow rate with and without LED information boards
Bottleneck at “Chofu Interchange Merging Area” (Data source: the station at 8.1kp detector)

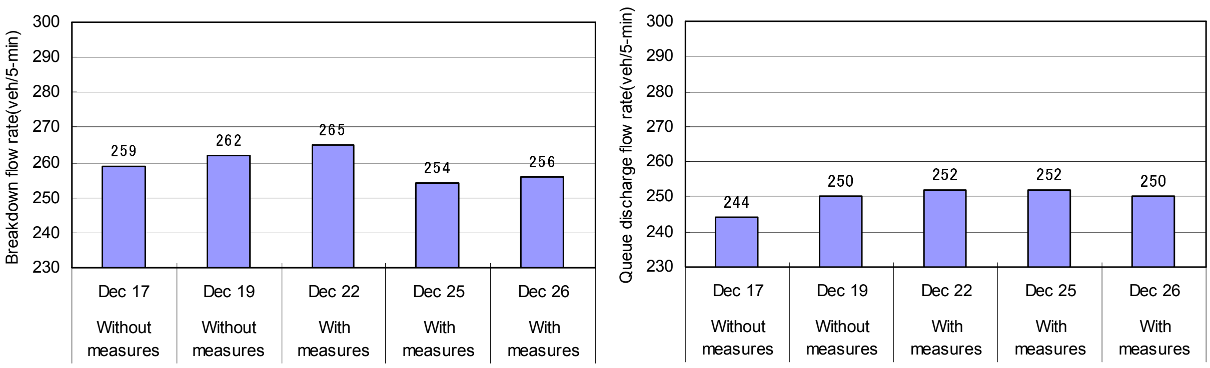



Figure 8: Changes in flow rate with and without LED information boards
Bottleneck at “JIndaiji Bus Stop Sag” (Data source: the station at 5.1kp detector)

### 5.4 Lane Utilization Rate Before Congestion With / Without LED Information Board

Lane utilization rate comparison was analyzed to understand how LED information boards is affected the distribution of traffic between lanes, compared to the case without LED information boards. The comparison of lane utilization rate between with and without LED information boards was undertaken along the study section. The data used for the analysis focuses on the maximum value of 5-min mean flow rates just before congestion obtained from detectors and video cameras.

Figure 9-11 presents an example of results when congestion occurs at “Chofu Interchange Merging Area”. As shown in the Figures, visible changes in the utilization rate between the station from 7.6kp to 6.8kp are due to the existence of the on- and off-ramp at Chofu Interchange. When the case without LED information boards (Figure 9) and with LED information boards (Figure 10, 11) were compared, the results show that the lane utilization rate with LED information boards drops in inner lane of the station at 6.8kp although the lane utilization rates in inner lane on the upstream shows high values as shown in Figure 10, compared to the ones shown in Figure 9. Moreover, the results also show that the decreasing tendency of the lane utilization rates in inner lane extends to the downstream.

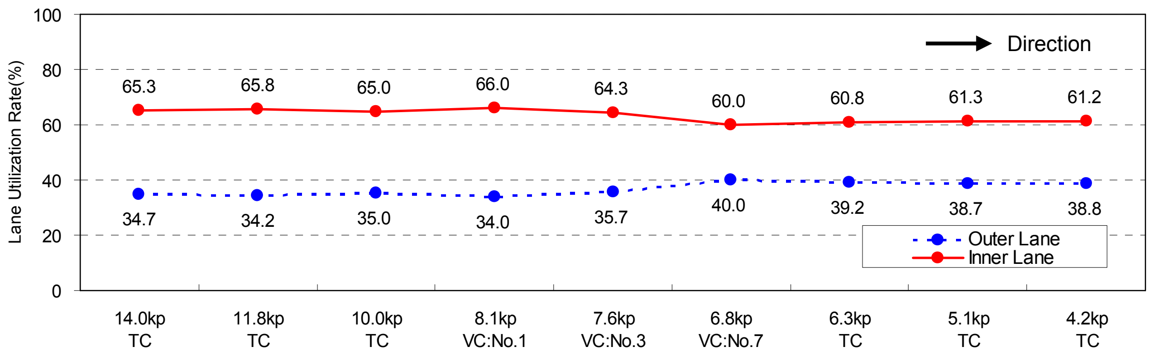



Figure 9: Spatial variance of lane utilization rate along the study section
(Data of 6:25 on Dec 15 without LED information boards)

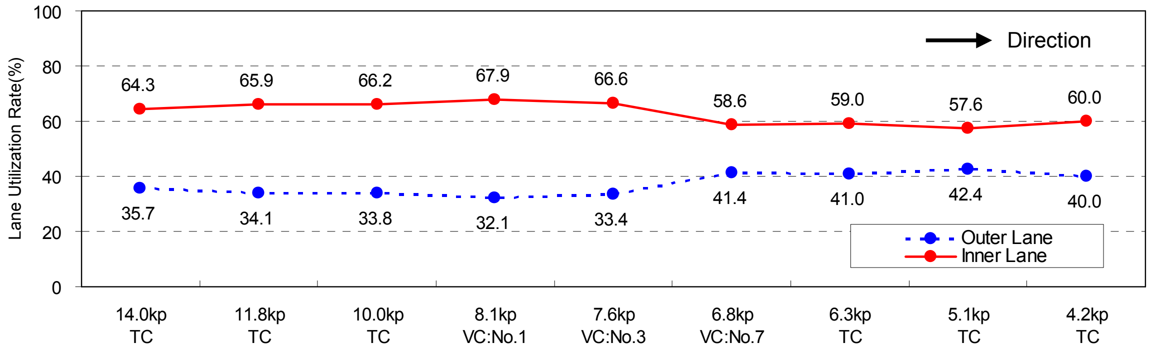



Figure 10: Spatial variance of lane utilization rate along the study section
(Data of 6:30 on Dec 16 with LED information boards)

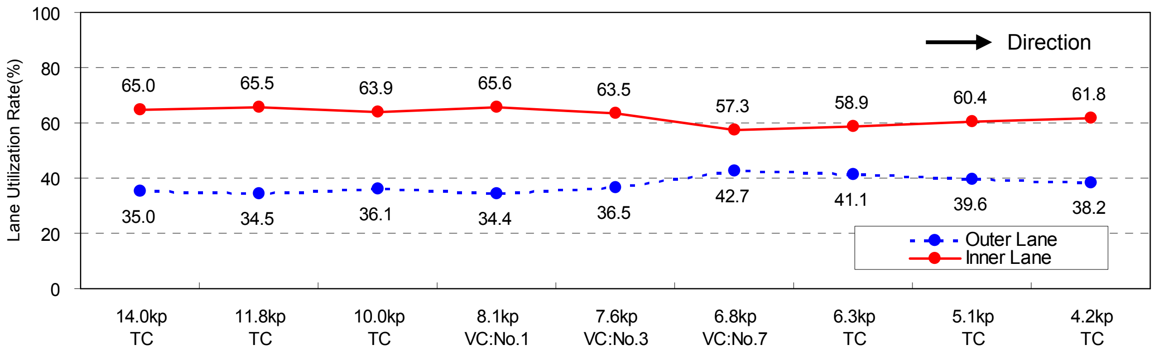



Figure 11: Spatial variance of lane utilization rate along the study section
(Data of 6:30 on Dec 18 with LED information boards)

Note: The station at 8.1kp is just a short distance downstream of the off-ramp at Chofu Interchange.
The station at 7.6kp is just a short distance upstream of the on-ramp at Chofu Interchange

### 5.5 Speed- Flow Relationship With / Without LED Information Board

The speed flow relationship with and without LED information board at a location just downstream of the section installed LED information boards, are shown in Figure 12. Data from the above period of analysis at location 5.1kp was used to represent the performance evaluation of LED information board. When comparing the flow/speed plots with and without LED information board, the following observations can be made:

- LED information board has slightly increased the breakdown flow rate in inner lane.
- LED information board also has slightly increased the queue discharge flow rate in both lanes.
- LED information board has resulted in a larger cluster of data points around 60km/hr speed level.
- LED information board also has slightly increased breakdown and queue discharge flow rate in outer lane.

The above observations have shown supporting evidence that LED information board increases flow rate of bottlenecks as shown in Figure 7-8 and improves the inequality of lane utilization as shown in Figure 9-11.

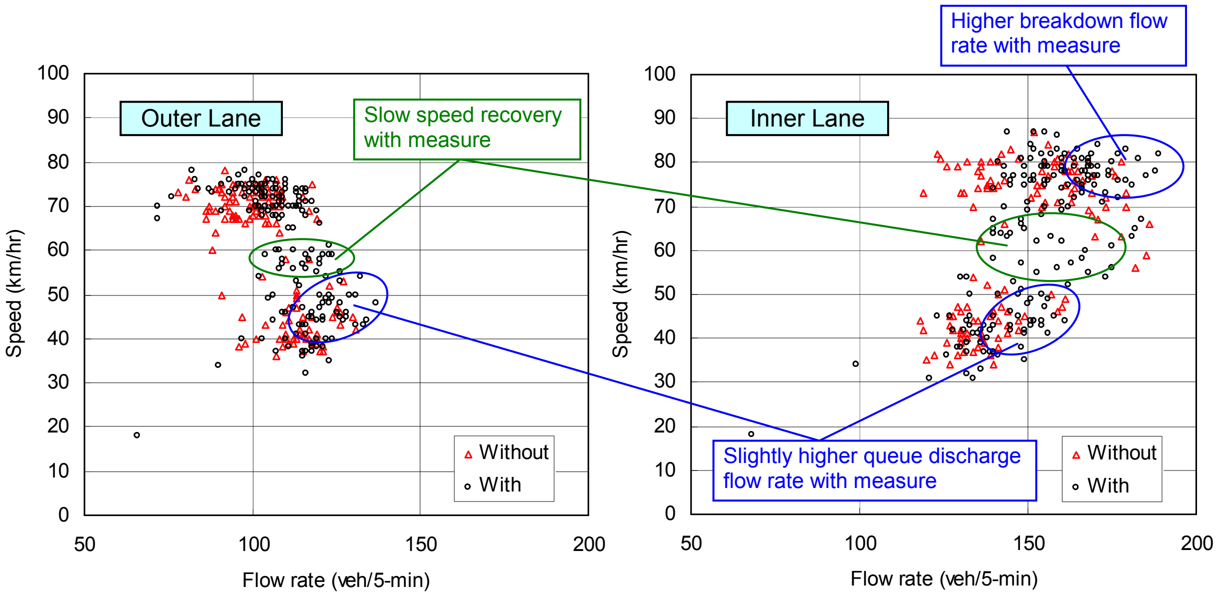



Figure12: Flow-speed relationship (Data source: the station at 5.1kp detector)

## 6. SUMMARY AND CONCLUSIONS

In the study, two types of experimental measures using LED information boards were carried out. One is the measure against “Chofu Interchange Merging Area” where LED information boards provide drivers with information to improve the lane utilization rates of both lanes from the upstream and the downstream in merging sections around Chofu Interchange. The other is the measure against “Jindaiji Bus Stop Sag” where drivers were provided with the information to urge them to avoid speed reduction or to accelerate.

The objective of the study was to evaluate the performance of the LED information board on traffic conditions, that is, flow rate, speed and lane utilization rate focusing on just before or after congestion. The main findings of the study can be summarized as follows:

- LED information boards have increased the breakdown and queue discharge flow rate at “Chofu Interchange Merging Area”. However, it was not clearly confirmed that the above two flow rates increased at “jindaiji Bus Stop Sag”.
- LED information boards have an effect on improving lane utilization rates. Moreover it was found that the improved lane utilization rate continues to the downstream of bottlenecks.
- LED information boards have recovered slow speed level in saturated traffic condition.

From the above-mentioned results, it was found that the measures using LED information boards had some effects on reducing congestion. However, it is conceivable that continuous investigations are necessary in the future because the data addressed in the study was limited.

## REFERENCES


[1] SAITO Y., XING J., NONAKA Y., ISHIDA T., UCHIYAMA H.(2005): An Experimental Study on Mitigating of Expressway Traffic Congestion with LED information Board, Proceedings of Eastern Asia Society for Transportation Studies, Vol.5, pp.919-928

[2] HARADA S., FUKASE M., MAEJIMA K., SATO H., SEKO K.(2007): Congestion Measures Effect of Expressway by Lane Availability Leveling, Journal of JSCE, Vol.42, No.5 (in Japanese)

[3] Nakatani R., Minagata T., Sato H., Ichikawa M.(2005): Evaluation on effect of provision of speed increase advice message by LED sign in consideration of time spent in queue, Proc. of the 25th JSTE Annual Conference (in Japanese)

[4] Xing J., Takahashi H., and Takeuchi T.,(2007): Mitigation of expressway traffic congestion with simple variable message sign, the 14th World Congress on ITS Beijing

[5] Central Nippon Expressway Co., Ltd., Homepage:
http://www.c-nexco.co.jp/info/others/pdf/20090423/20090423_01.pdf (in Japanese)

[6] East Nippon Expressway Co., Ltd., Homepage:
http://www.e-nexco.co.jp/pressroom/press_release/head_office/h18/0908/ (in Japanese)

# Evaluation of a Fog Detect & Warning System Using a Vehicle Simulator

SOO-YEON KIM, SUK-KI LEE,JUN-HWA Jeong, JONG MIN KIM
Researcher
Korea Institute of Construction Technology
1190, Simindae-Ro, Ilsanseo-Gu, Goyang-Si
Gyeonggi-Do, 411-712, Republic of Korea
Fax: +81-31-910-0089
E-mail: miloo21@kict.re.kr

**ABSTRACT**

While the difference in temperatures at the earth's surface between day and night leads to flatland fog from night to early morning, mountain fog sets in when damp air rises along the slope of mountains. In a foggy day, visible distances depend greatly on the ever-changing concentration of fog. Most of chained rear-end collisions in a foggy day are caused by significant differences in speeds between vehicles. Then, it is important to minimize the collisions in fog. Fog Detect & Warning System (FDWS) was conceived to provide safety operating speeds and inter-vehicle safety distances to drivers in a foggy day through text messages, color codes, and others.

The objective of this study is to estimate the effects of FDWS on drivers' safety behaviors with a vehicle simulator and a questionnaire survey. A roadway section 1 km long with bidirectional 2 lanes was chosen for the estimation efforts. The test segment was configured from the Munsan to the Jeokseong county in National Highway 37. Scenarios associated with fog were set as heavy fog (a visible distance of 30 m) and light fog (a visible distance of 80 m). Also, the distance between driver warning lamps was considered by 25 m and 30 m, respectively. The experiment conducted in this study employed the total of 31 adults who were randomly selected from 20- to 60-year old. Before performing the experiment, each subject had enough times to be familiar with the vehicle simulator and was instructed by the guideline of the experiment. After performing the main test, a questionnaire survey was undertaken for each subject.

It is expected that FDWS will be implemented in National Highway 37 after determining if FDWS appears to have safety effects with performing complementary and inspectional steps. A follow-up study will be conducted to estimate actual effects on which FDWS has on driver's behaviors in a pilot roadway.

## 1. Introduction

Fog that is familiar with us is generated from night to early morning due to the difference in temperature at the earth’s surface between day and night. Mountain fog is generated when damp air rises along the slope of mountains regardless of specific conditions like day and night. Because visible distances in fog can be varied by the concentration of fog, the fog significantly affects safe driving. For the actual figure in traffic collisions according to the condition of weather, although the number of collisions in fine days increased compared to other weather conditions, Foggy days have average fatal collision rates of 18.8 collision year, which fine days have average fatal collision rate of 12.2 collision year. Also, the foggy days appear of have fatal collision0 rates than grey and rainy days. In spite of all the danger in foggy roads, the facilities that guarantee visible distances in fog are insufficient in Korea, and road signs and information providing systems are also unsatisfactory. In addition, fatal collisions caused by fog have been increased due to the lack of the proper application based on the experience of road manages in the design and installation of facilities at the early stage of these processes.

Thus, this study develops a Fog Detect & Warning System (FDWS) and evaluates the effects of this system with a driving simulator. Fillally, this study is aimed at conceiving the way of develops making vehicles safely in foggy days through installing the system in actual roads.

## 2. Tendency in the traffic collision by weather conditions

According to the data presented by the Road Traffic Authority (Statistics on traffic collisions in 2008) that represents the tendency in traffic collisions during the last six years according to weather conditions except for unclassified and unidentified cases, a large part of collisions, about 83%, occurred in fine days, and the second highest rate was 9.39% in rainy days, 6.42% in grey days, 0.95% in snow days, and 0.21% in foggy days. In the fatal collision rate in these collisions according to weather conditions, fine days, clouded days, rainy days, foggy days, and snow days showed 2.76%, 4.16%, 3.61%, 13.41%, and 2.99%, respectively, in which the foggy days represented the highest fatal collision rate in these conditions. It can be seen that a certain abnormal weather condition, such as a foggy day, may bring huge collisions like chained rear-end collisions and large number of victims in such collisions.

Table 1 Tendency in the traffic collisions by weather conditions (Unit: cases)

| Weather Condition Year | Sum | | Fine Days | | Grey Days | | Rainy Days | | Foggy Days | | Snow Days | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Cases | Fatal | Cases | Fatal | Cases | Fatal | Cases | Fatal | Cases | Fatal | Cases | Fatal |

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 2002 | 231,026 | 7,222 | 207,778 | 6,151 | 9,176 | 376 | 12,548 | 596 | 423 | 40 | 1,101 | 59 |
| 2003 | 240,104 | 7,212 | 181,503 | 5,026 | 23,127 | 869 | 32,439 | 1,122 | 191 | 96 | 2,844 | 99 |
| 2004 | 220,755 | 6,563 | 182,021 | 5,145 | 14,133 | 594 | 21,463 | 705 | 606 | 61 | 2,532 | 58 |
| 2005 | 212,712 | 6,341 | 179,009 | 5,019 | 12,881 | 582 | 17,361 | 619 | 404 | 44 | 3,057 | 77 |
| 2006 | 212,400 | 5,700 | 177,447 | 4,328 | 12,487 | 554 | 20,168 | 706 | 566 | 75 | 1,732 | 37 |
| 2007 | 210,560 | 6,153 | 174,561 | 4,733 | 13,405 | 570 | 20,704 | 751 | 562 | 53 | 1,328 | 46 |
| Total | 1,327,557 | 39,191 | 1,102,319 | 30,402 | 85,209 | 3,545 | 124,683 | 4,499 | 2,752 | 369 | 12,594 | 376 |

### 3. Literature reviews

In the handbook of traffic safety facilities (National Police Agency, 2000), a warning sign of fog is to apply to roads that appear to have particular weather conditions like a heavy fog. Also, information on fog and speed limitations are provided to drivers by the Variable Message Sign (VMS) that is installed as a part of traffic management systems.

Fig. 1 Warning sign in fog areas


Fig. 2 Electronic display board

In addition, lights that show no scattering within a specific angle are installed at a position below 2m in order to guarantee a visible distance of drivers in frequent fog areas.

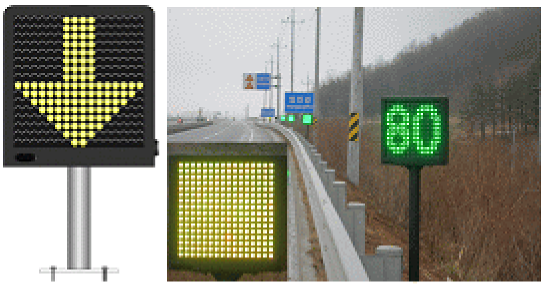
Also, fog warning systems that inform precaution on fog to drivers using flash lights and horns and lane departure warning systems, such as bumpy road pavement or projected lane installed on a road shoulder, that stimulate awareness on lane departure to drivers are used for such particular conditions.

In the case of other countries, Cooper & Sawyer (England, 2005) installed a fog warning system in highways and investigated its effects in the study on fog warning systems. They installed 54 fog detectors in fog areas. In the case of the visible distance that is presented by below 250m, a “FOG” message was presented at the VMS, which was installed at 0.8 ~ 2.2km ahead in a fog area. It represented no evidence of safety effects in the VMS warning message for good load conditions. However, in the case of bad road conditions with curved sections, it showed that drivers well obeyed to the VMS warning message. In the comparison between the VMS message with 80km/h and the VMS+"slippery road" symbol with 60km/h, the average speed decreased by 2.5km/h, and the inter-vehicle distance determined by below 1 second showed a decrease in the speed by 18%. Thus, a method that applies both the VMS message with speeds and the warning message, which shows the conditions of forward areas, is a more effective way than that of regular fixed signs.

### 4. Fog Detect & Warning System (FDWS)

The FDWS presents proper driving speeds and inter-vehicle distances to drivers using numbers or surface emission devices in order to prevent traffic collisions caused by over-speed driving and lack of inter-vehicle safety distance in fog areas.

The installation distance of the Light Bar Type1 is determined by 25m or 30m in which the Light Bar Type1 consists of specific warning devices as shown in Fig. 3 for maintaining safety driving speeds and inter-vehicle distances in fog areas.



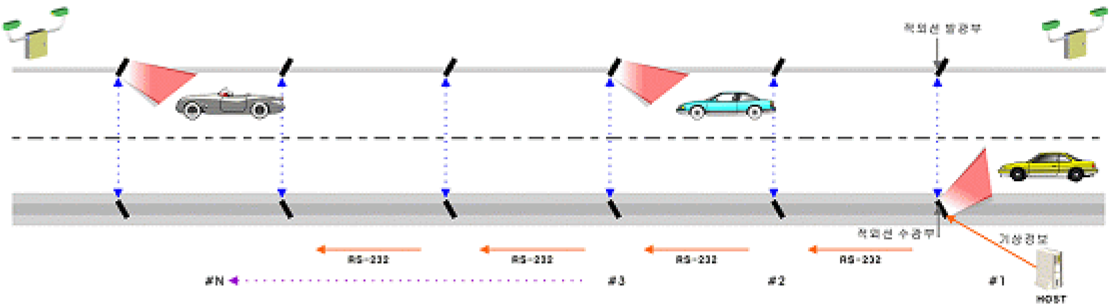



Fig. 3 Configuration of the Light Bar (Type 1)

The installation distance of the Light Bar Type2 is installed at the internals ofby 25m or 30m in which the Light Bar Type2 notifies the presence of vehicles ahead of drivers in fog areas by presenting the location of forward vehicles. It shows a high visibility with high brightness LEDs, which are used as the major light source and make possible to see objects from a distance. Also, in the case of the auxiliary light source, it uses low brightness LEDs to minimize the blinding of drivers and adds a surface emission function for playing a role in guidance lights.

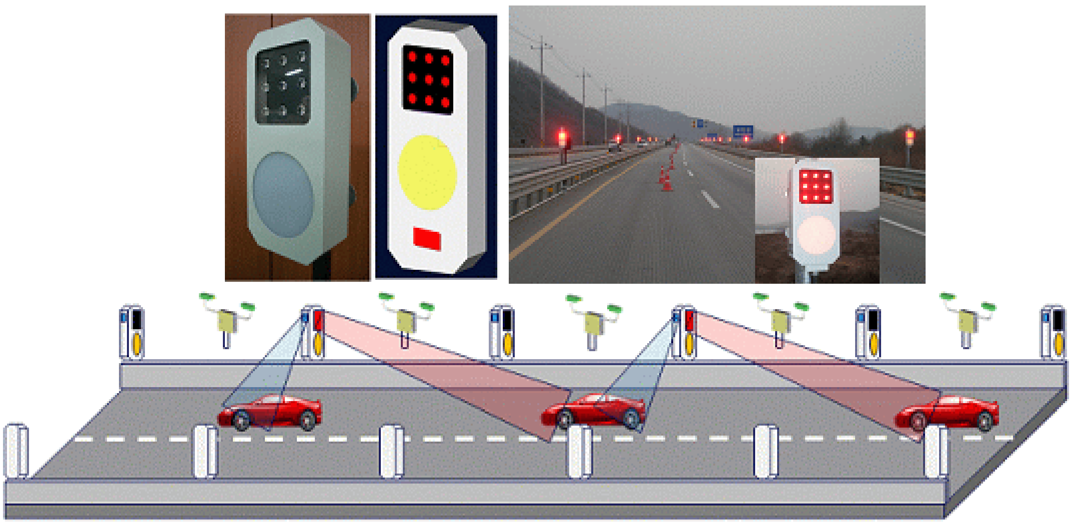


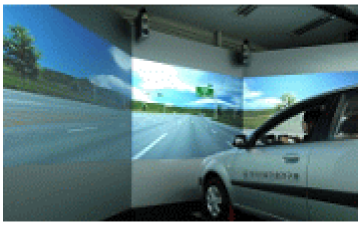
Fig. 4 Configuration of the Light Bar (Type 2)

**5. Virtual driving test**

A test segment used in the virtual driving test was selected as a 1km long segment around the Nulnochun bridge in National Highway 37 that is a bidirectional road with two lanes from Munsan to Jeokseong. The objective of this test is to analyze proper driving speeds and inter-vehicle distances according to visible distances in fog areas.


Fig. 5 Test segment

The road simulator, K-ROADS, developed by the Korea Institute of Construction Technology was used to evaluate the effietiveness of FDWS of drivers during the driving based on the FDWS. Fig. 6 represents the K-ROADS, and Fig. 7 and Fig. 8 show the computer graphic images performed by using a virtual reality in fog areas.

| | | |
|---|---|---|
|  |  |  |
| Fig. 6 K-ROADS | Fig. 7 Type 1 | Fig. 8 Type 2 |

(1) Summary of the test

- Test segment: National Highway 37, Munsan-Jeoksung, 1km long segment around the Nulnochun bridge
- Subjects: 30 adult males and females (corrected eye sight more than 0.7, actual driving experience more than 1 year with a driver's license)
- Scenario
  - ① Time of the virtual road: Early morning in a frequent fog road
  - ② Fog condition: Thick fog (visible distance: 30m), thin fog (visible distance: 80m)
- Condition of the TYPE 1: No precedence vehicles, yellow limit speed+surface emission effects
- Condition of the TYPE 2: Notifying vehicles to rear vehicles by using the major light source through lighting it after detecting vehicles
- Test procedure: Introduction to the test, training for the simulator, and explaining the

subject and objective of the test including details in the test

- FDWS driving test: Driving as a usual manner and obtaining driving data from the test
- Subjective survey: After completing all driving tests, a questionnaire survey is undertaken for each subject in order to investigate the subjective psychological state of the participants

(2) Analysis of the driving data

In the analysis of driving patterns, the driving patterns were first investigated using the driving speed and inter-vehicle graphs for the test segment. Second, the difference in the response rate of drivers for the installation distance of warning lights was analyzed. Then, a general accommodation rate was analyzed for the system. Based on the results of the analysis, it was analyzed that whether the system contributes to the visibility and safety driving for drivers.

(2-1) Obtained data

Light bar(Type 1)

In the driving test for 31 subjects, a total of 31 individual speed data were obtained by visible distances as noted in Table 2.

Table 2 Obtained data in the Light bar (Type 1)

| Light bar (Type 1) | | Number of available data for the analysis | Number of lost data | Total number of data |
|---|---|---|---|---|
| Visible Distance: 30m | Installation distance: 25m | 31 | 0 | 31 |
| | Installation distance: 30m | 31 | 0 | 31 |
| Visible Distance: 80m | Installation distance: 25m | 31 | 0 | 31 |
| | Installation distance: 30m | 31 | 0 | 31 |

② Light bar(Type 2)

In the driving test for 31 subjects, the speed data and inter-vehicle data that can be analyzed according to visible distances are summarized in Table 3. The criteria for removing data can be determined as follows: first, there are no followings to precedence vehicles; second, the behavior of a lead vehicle comes into or out the middle of the analysis segment; and third, the difference in driving speeds between the maximum and the minimum values in an analysis section represents

more than 20km/h. Then, the data corresponded to these criteria was removed as improper data.

Table 3 Obtained data in the Light bar (Type 2)

| Light bar (Type 2) | | Number of available data for the analysis | Number of removed data | Total number of data |
|---|---|---|---|---|
| Visible Distance: 30m | Installation distance: 30m | 13 | 18 | 31 |
| | Installation distance: 25m | 16 | 15 | 31 |
| Visible Distance: 80m | Installation distance: 30m | 8 | 23 | 31 |
| | Installation distance: 25m | 17 | 14 | 31 |

(2-2) Analysis of the driving patterns of drivers in the FDWS

① Light bar(Type 1)

Driving patterns of the subjects were analyzed according to given fog conditions. In the thick fog condition with the visible distance of 30m and speed limit of 30km/h, subjects increased their driving speeds and decreased speeds about 10km/h after passing a distance of 1650m in which the system is installed. In the case of the thin fog condition with the visible distance of 50m and speed limit of 50km/h, there were no differences in driving speeds at the point of 1650m where the system was installed before and after passing the point.

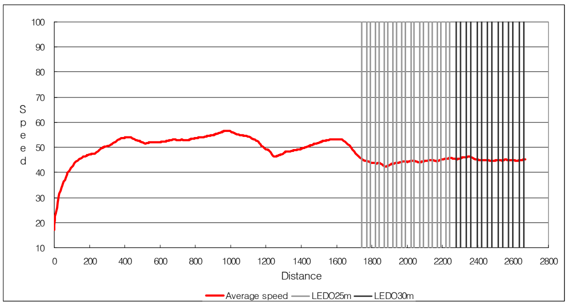



Fig. 8 Light bar

(Type1: Visible Distance - 30m)

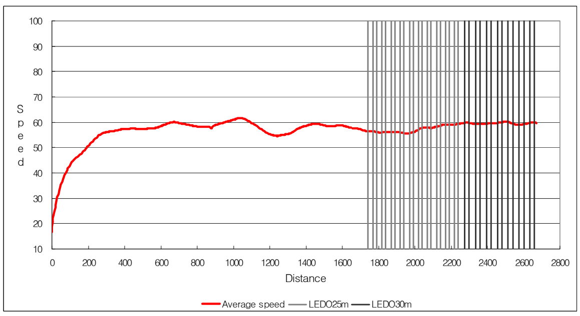


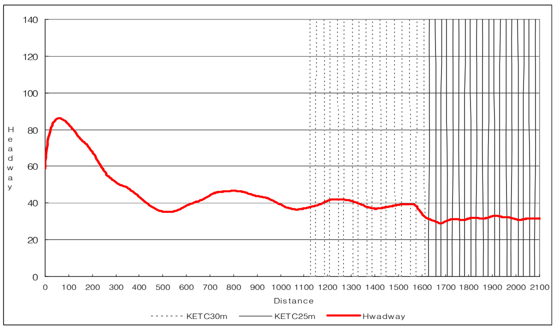
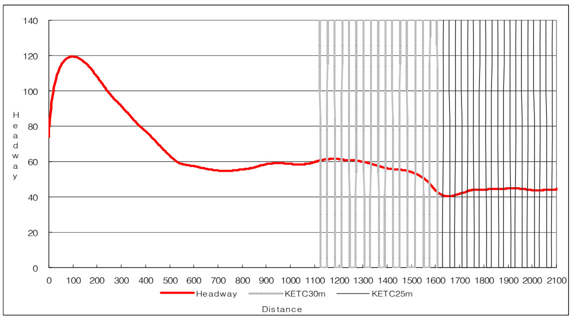

Fig. 9 Light bar

(Type1: Visible Distance - 80m)

② Light bar(Type 2),

In the investigation of the driving patterns of drivers using inter-vehicle distances for each fog condition, the driving patterns for the thick fog condition with the visible distance of 30m and speed limit of 30km/h and the thick fog condition with the visible distance of 50m and speed limit of

50km/h represented similar results.

| | |
|---|---|
|   |   |
| Fig. 10 Light bar<br>(Type 2: Visible Distance - 30m) | Fig. 11 Light bar<br>(Type2: Visible Distance - 80m) |

(2-3) Analysis of the installation distance of the warning lights for drivers in the FDWS

① Light bar(Type 1)

For the Light bar (Type 1), tests on the difference in the average speed in test sections that was caused by the installation distance of the warning lights for drivers in which the lights were installed with the intervals of 25m and 30m were verified under the conditions of the visible distance of 30m and speed limit of 30km/h and the visible distance of 80m and speed limits of 50km/h. Since the sample data did not satisfy the normality, a non-parametric statistical test method was used. The hypothesis used in this test can be determined as follows:

$$H_0 : \mu_{25} = \mu_{30} \qquad\qquad H_1 : \mu_{25} \neq \mu_{30}$$

Since the significant probabilities in the two cases with visible distances of 30m and 80m were 0.000 and that were smaller than the value of α=0.05, the null hypothesis that determines the same average driving speed for the installation distances of 25m and 30m was rejected. It is considered that the average driving speeds in installed sections are different. However, it is also difficult to regard which installation distance, 25m or 30m, is safe in driving only based on the results of the non-parametric statistical test. It is important to perform even driving including the low driving speed in fog conditions. Thus, this study investigated the distribution of the samples according to the installation distances. The speed distributions in the installation distance of 25m and 30m were 4.41 and 0.21, respectively, in which the installation distance of 30m represented a smaller distribution than that of the distance of 25m. It seems that drivers in the installation distance of 30m showed more even driving behaviors than the installation distance of 25m. However, with the rescues it is necessary to consider the results obtained from the adaptation in the system through the driving.

② Light bar(Type 2)

It is necessary to establish a reference for the minimum inter-vehicle distance for the safety of drivers in foggy days because the Light bar (Type 2) evaluates the effects of the system based on the inter-vehicle distance differed from the Light bar (Type 1).

Thus, this study deduced the minimum inter-vehicle distance using a stopping sight distance formula. In the assumption in which the braking distance of a precedence vehicle during the driving shows the same braking distance as of following vehicles, the inter-vehicle distance was calculated by considering the free running distance only except for the braking distance, and the time (t) in the free running distance was obtained as an inter-vehicle distance per speed through applying the recognition time of 2.5 seconds.

$$\text{minimum headway distance} = \text{speed}(\frac{\text{km}}{\text{h}}) \times \frac{1}{3.6} \times 2.5\text{s}$$

The test groups were classified into two groups. One does correspond to the minimum inter-vehicle distance, and the other does not for the data that was to be analyzed by using the deduced equation.

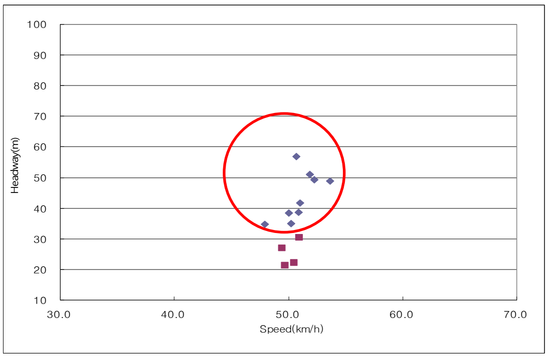



Fig. 12 Installation distance - 30m (visible distance: 30m)

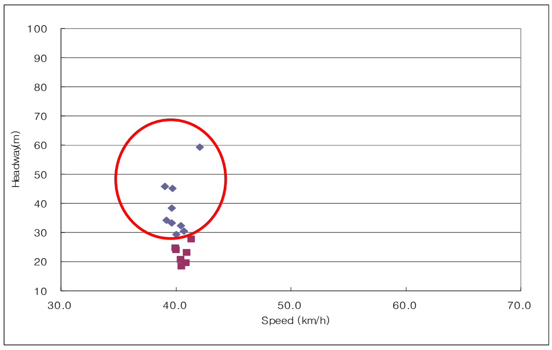



Fig. 13 Installation distance - 25m (visible distance: 25m)

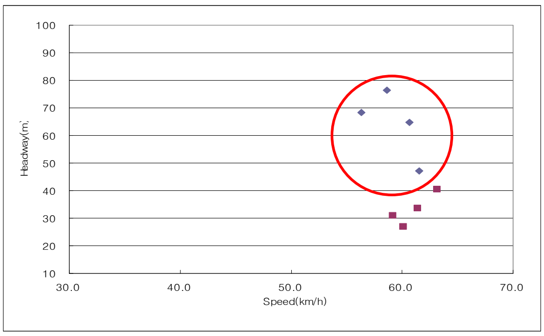



Fig. 14 Installation distance - 30m (visible distance: 80m)

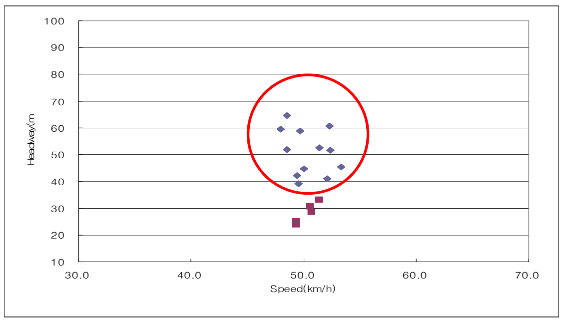



Fig. 15 Installation distance - 25m (visible distance: 80m)

This study compared and analyzed the data that maintained the minimum inter-vehicle distance in order to statistically investigate the difference in the installation distances of the warning lights for

drivers according to the visible distance in the Light bar (Type 2).

In the test of the data, an independent sample test was used. Also, in the test of the normality of the samples before applying the test, the analysis was applied because all the results were satisfied. The hypothesis on the independent sample test was determined as follows:

$$H_0 : \mu_{25} = \mu_{30} \qquad\qquad H_1 : \mu_{25} \neq \mu_{30}$$

An assumption that showed the same distributions in two samples because the significant probability in an equivalent distribution test represented 0.73 and 0.64, respectively, in the visible distances of 30m and 80m and that was larger than the significant level, α=0.05 was established. Also, in the results of the independent sample test, because the significant probability in the case with the visible distance of 30m showed 0.23 and that was larger than the significant level, α=0.05, it can be seen that there were no differences in the results due to the installation distance. In the condition with the visible distance of 80m, because the significant possibility in an independent sample test represented 0.03 and that was smaller than the significant level, α=0.05, there were certain differences in the results due to the installation distance. However, it is necessary to reflect the lack of reliability in the results of the analysis due to the small number of data that are to be analyzed compared to that of the case with the visible distance of 30m.

(2-4) Accommodation rate of the FDWS

① Light bar (TYPE 1)

This study deduced the accommodation rate of the system based on the subjects related to the decrease in driving speeds, which showed more than 5km/h, compared with the initial driving speed after the experience of the FDWS in fog roads through the simulator. The deduced accommodation rate can be determined as follows:

- For the visible distance of 30m: The accommodation rate was 71% by the margin of error as ±0.16 at the confidence level of 95%.
- For the visible distance of 80m: The accommodation rate was 45% by the margin of error as ±0.18 at the confidence level of 95%.

② Light bar (TYPE 2)

This study deduced the accommodation rate of the system based on the subjects who maintained the minimum inter-vehicle distance in the FDWS in fog roads. The deduced accommodation rate can be determined as follows:

- For the visible distance of 30m: It was investigated that the accommodation rate of the inter-vehicle distance of the subjects showed 69% by the margin of error as ±0.25 at the confidence level of 95% for a total of 13 data in the installation distance of 30m. In the case of the installation distance of 25m, the accommodation rate showed 56% by

the margin of error as ±0.24 at the confidence level of 95% for a total of 16 data.

- For the visible distance of 80m: It can be considered that the accommodation rate in the installation distance of 30m showed 50% by the margin of error as ±0.34 at the confidence level of 95% for a total of 8 data. Also, in the case of the installation distance of 25m, the accommodation rate represented 71% by the margin of error as ±0.21 at the confidence level of 95% for a total of 8 data.

In the comparison of the difference in the characteristics of fog between the simulator and the actual road, it can be seen that the fog experienced in the simulator may affect the driver's behavior in actual fog roads through considering the accommodation rate obtained in the test.

In the comparison of the system accommodation rates in the Type 1 and Type 2, the accommodation rate of the Type 2 that represents inter-vehicle distances showed higher rates than that of the Type 1 that shows speed limits. It shows that the system that guides the inter-vehicle distance may help the safe driving of drivers compared to the system that guides speed limits in fog roads.

## 6. Conclusion

This study proposed the FDWS to guide safe driving by maintaining driving speeds and inter-vehicle distances in fog roads and verified the effects of this system through some experiments with a simulator. This study first investigated general driving patterns and analyzed the differences in the speeds of the warning lights for drivers and the inter-vehicle distances in the system. Then, this study investigated whether the drivers accommodated to the system after experiencing this system.

As a result, there were no specific differences in driving speeds and inter-vehicle distances according to the installation distances of warning lights. Also, it was considered that the system was effective for fog conditions because the Type 1 and Type 2 employed in the FDWS represented high accommodation rates more than 50%.

In future studies, the effects of the system will be verified by some drivers in actual fog conditions by installing practical samples to a TEST BED based on the results of the experiments performed in the simulator after compensating the reliability through expanding the number of subjects.

ACKNOWLEDGEMENTS

This research was supported by the grant (06 C01) from the implementation of more Efficient Transportation System Program funded by the Ministry of Land, Transport and Maritime Affairs of the Korean government.

## References

1. AASHTO, A Policy on Geometric Design of Highways and Streets, 1994.
2. Alexander, G.J., and H. Lunenfeld, "Driver Expectancy in Highway Design and Traffic Operation," Federal Highway Administration, Report No, FHWA-TO-86-1, Springfield, Virginia, 1986.
3. J. E, Leisch and J. P Leisch, "New Concepts in Design-Speed Application", TRR 631, 1977.
4. Ruediger Lamm, Basil Psarianos, & Theodor Mailaender, "Highway Design and Traffic Safety Engineering Handbook", McGrawHill. 1999.
5. Sara Nygardhs, VMS-Variable Message Signs, VTI Report 570A, 2007.
6. Myra Blanco and Jonathan M. Hankey, Visual Performance During Nighttime Driving in Fog, FHWA-HRT-04-137, 2005.
7. Virpi Anttila, Visual demand of bilingual message signs displaying alternating text messages, Technical Research Centre of Finland, Transoprtation research Part F3 65-74, 2000.
8. Gudmundur F. Ulfarsson, The effect of variable message and speed limit signs on mean speeds and speed deviations, Int. J. Vehicle Information and Communication Systems, Vol. 1, Nos. 1/2, 2005.

# Classification of Highway Curve Patterns and Its Use for More Accurate Crash Predictions

Jaisung CHOI
Professor
Department of Transportation Engineering
University of Seoul
90 Jeonnong-dong, Dongdaemun-gu
Seoul, South Korea
Fax: +82-2-2210-2653
E-mail: traffic@uos.ac.kr

Sangyoup KIM
Graduate Student
Department of Transportation Engineering
University of Seoul
90 Jeonnong-dong, Dongdaemun-gu
Seoul, South Korea
Fax: +82-2-2210-2653
E-mail: road@uos.ac.kr

Kyungsung HWANG
Graduate Student
Department of Transportation Engineering
University of Seoul
90 Jeonnong-dong, Dongdaemun-gu
Seoul, South Korea
Fax: +82-2-2210-2653
E-mail: Jordanhks@hanmail.net

**Abstract:** This paper presents an improved crash prediction method that classifies highway curves into several patterns including tangent, single curve, compound curve, and reverse curve. This classification presupposes that a crash is likely to occur being influenced not only by the curve itself but also by the curve surrounding conditions, and this view was tested by applying freeway crash data collected in South Korea to the regression analysis. As a result, it was found that: (1) longer than 2.4km tangent sections were unsafe, (2) single curve sections should be designed with longer radii and lengths to reduce crashes, (3) compound curve section and reverse curve section involved reduced crashes when two consecutive curves have little curve radius change, and (4) fewer crashes were observed when vertical grade values were small. It is believed that this research approach was very effective in explaining crash occurrences for selected freeways, and its wider use is recommended for more accurate crash predictions.

***Key Words:*** *Highway Curve Patterns, Crash Model, Continuous Analysis, Regression Model, Highway Consistency*

## 1. INTRODUCTION

### 1.1 Background and Scope

Motor vehicle accidents continue to be a leading cause of death and injury, and extensive effort has been made by many researchers to explain traffic accident occurrence and factors affecting accidents. The researchers developed various types of model to characterize each crash and made use of their expertise obtained in the model development to accurately predict future accidents. Their predictions can be excellent, but often are poor. How does this

happen? Actually, it is safe to say that no one can predict with great certainty the exact locations and types for all crashes. Nonetheless, the researchers believe that their crash predictions can be improved significantly by applying better analytical skill. For example, Lee and Mannering (1999) stated that applying traffic volume as the exposure in crash analysis might involve prediction error. According to Lee and Mannering (1999), there exists a U-shaped correlation between traffic volume and crash probability. Thus, by modeler's right treatment of traffic volume in the exposure variable, their predictions can be greatly improved. Another example was presented by Jovanis and Chang (1986). They discovered that as vehicle kilometer travelled increases, so does the variance of the accident frequency. Also they found that linear regression is not restrained from predicting negative accident frequency. Apart from these literature review findings, the authors found during our accident modeling process that engineers seemed to make mistakes in determining the highway length subject to analysis. Let us consider Figure 1 that is a conceptual diagram of a highway segment where an accident occurs. Usually engineers determine accident rate based on Area 'A' highlighted in Figure 1. However this length application is wrong because accidents occur being influenced not only by isolated highway geometrics in the accident site but also by continuous highway geometrics that include the upstream segment. This problem must be resolved to get reliable crash predictions, and this research attempts to characterize how surrounding areas for a crash site influence the development of the accident model. A more engineering oriented crash prediction method assuming that a crash is likely to occur being influenced by both the curve itself and the surrounding continuous curve geometric conditions is adopted in this research.

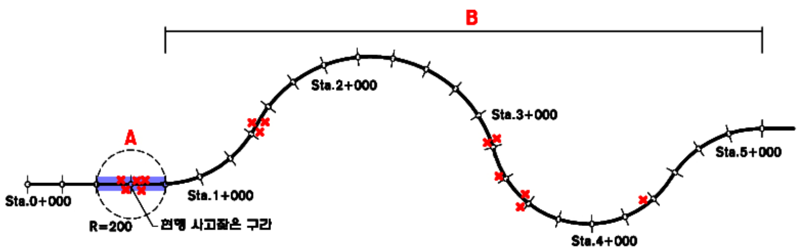



Figure 1 Isolated Analysis of Highway Curves

### 1.2 Research Approach

This research uses the following approach. First, considering the effects on crash occurrences of different highway curve settings, the research classifies horizontal curve patterns into a few distinctive types. Second, crash data and the as-built plan and profile drawings are integrated into a single data base. Third, statistical analysis is done to investigate the relationships between different highway curve settings and crash occurrences.

## 2. CONTINUOUS HIGHWAY SEGMENT AND ANALYSIS

This research underscores the importance of applying the consistency concept in geometric design, and for the most part tries to apply it to develop more accurate highway curve crash models. The essence of the consistency concept in geometric design is to avoid abrupt speed changes in operating speeds, and the consistency concept became an important design issue since 1960s (2). In this research, it is hypothesized that this speed change would also have

substantial impacts on crashes, and the speed change would be in return influenced by the upstream and downstream geometric conditions of a curve. Therefore, instead of applying the isolated geometrics of a curve, the continuous highway geometric condition as presented in Figure 2 is applied in this research to develop more reasonable and accurate crash models.

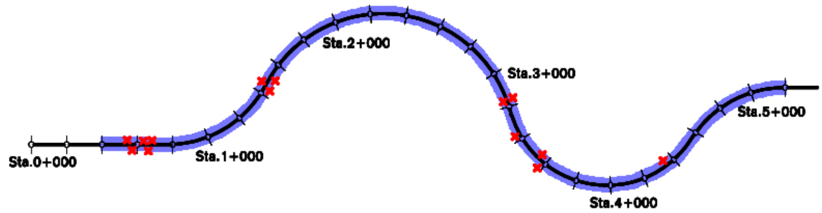



Figure 2 Continuous Analysis of Highway Curves

## 3. DATA BASE DEVELOPMENT

The data base for this research for the most part consists of 9 four lane freeway segments in South Korea. Total length is 1,399.8 km, and geometric conditions and crash characteristic data are provided by the Korea Highway Corporation and integrated into a primary data base for this research. Table 1 summarizes the list of freeways involved in this research.

Table 1 Freeway List for this Research

| No. | Freeway Name | Opening Date | Segment | Length(km) |
|---|---|---|---|---|
| 10 | Namhae Freeway | 1973.11.14 | Soonchun ~ Masan | 127.1 |
| 15 | WestSea Freeway | 2001.12.21 | Mokpo ~ Seocheon | 160.9 |
| 20 | Iksan-Pohang Freeway | 2004.12.8 | Dodong JCT ~ Pohang IC | 68.4 |
| 25 | Honam Freeway | 1973.11.14 | Soonchun ~ Nonsan | 194.2 |
| 35 | Daejon-Jinju Freeway | 2001.11.29 | Jinju ~ Biryong | 215.3 |
| 45 | Jungbu Inner-side Freeway | 2004.12.15 | Masan ~ Yeoju | 203.6 |
| 50 | Yeongdong Freeway | 1975.10.14 | Saemal ~ Gangneung | 91.5 |
| 55 | Central Freeway | 2001.12.19 | Geumho ~ Chuncheon | 278.6 |
| 65 | Donghae Freeway | 1975.10.14 | Donghae ~ Gangneung | 60.2 |

The KHC provided the freeway geometric condition in CAD form for this research. However, the provided data were in fragmented forms, and this research has to combine them with surrounding geometric condition so as to get the continuous characteristics of highway curves and their associated crash patterns. The followings are the highway geometric elements extracted for this research.

- Tangent: length
- Curve: radius, length
- Transition Curve: clothoid parameter (A), length
- Vertical: grade (upgrade, downgrade)

This research acquired initial crash data from the KHC for three years 2003-2005, but this data was intended to be applied for the isolated analysis of highway curve shown in Figure 1. Therefore, an additional effort was needed to match them to the continuous analysis of highway curve. This required lots of laboratory work, because it was needed to extract 2,540 highway geometry related crashes out of the total of 9,707. Then a schematic arrangement for linking the tangent and curve characteristics to freeway crash occurrences was made in this research. Table 2 and Table 3 summarize the result.

Table 2 Linking Highway Geometric Conditions to Crash Occurrences (Tangent Section)

| Tangent Length | No. of Section | Ave. Length(m) | AADT(veh/day) | Ave. Accident Rate (100mil.veh.km) |
|---|---|---|---|---|
| 0~500 | 658 | 247 | 31,018 | 5.60 |
| 500~1000 | 395 | 730 | 29,763 | 4.77 |
| 1000~1500 | 167 | 1,240 | 25,893 | 3.92 |
| 1500~2000 | 82 | 1,734 | 28,646 | 3.95 |
| 2000~2500 | 52 | 2,264 | 18,370 | 2.54 |
| 2500~3000 | 26 | 2,747 | 21,388 | 3.67 |
| 3000~3500 | 14 | 3,170 | 17,854 | 4.18 |
| 3500~4000 | 6 | 3,756 | 20,689 | 5.70 |
| 4000이상 | 4 | 4,569 | 19,689 | 7.81 |

Table 3 Linking Highway Geometric Conditions to Crash Occurrences (Curve Section)

| Tangent Length | No. of Section | Ave. Length(m) | AADT(veh/day) | Ave. Accident Rate (100mil.veh.km) |
|---|---|---|---|---|
| 0~500 | 80 | 463 | 38,832 | 9.56 |
| 500~1000 | 450 | 673 | 32,457 | 5.01 |
| 1000~1500 | 359 | 833 | 29,429 | 4.31 |
| 1500~2000 | 129 | 1,101 | 28,942 | 3.28 |
| 2000~2500 | 204 | 973 | 28,618 | 3.73 |
| 2500~3000 | 55 | 1,102 | 22,698 | 7.16 |
| 3000~3500 | 154 | 847 | 22,101 | 3.78 |
| 3500~4000 | 30 | 992 | 21,516 | 6.97 |
| 4000이상 | 175 | 778 | 21,819 | 4.07 |

## 4. DEVELOPMENT OF MODELS

### 4.1 Classification of Highway Geometric Conditions

In crash analysis, it is important to apply an appropriate amount of highway unit length. This length separates one highway length from others, and if too short a length is applied, the effect of surrounding highway conditions might be excluded, and if too long, the detailed local crash site effect can be excluded. In this regard, this research decided to use approximately 300 meters as the unit highway length, based on 120 km/h design speed, pre-maneuver time 4 seconds, and driver deceleration 3.4 m/sec/sec. Another important highway length criterion applied in this research was the a priori highway length that is the upstream highway length from a crash site where the motorist travels and approaches to the crash site, resulting in varying degrees of crash severity or crash patterns depending upon how motorists absorb the geometric characteristics associated with the upstream highway segment. In order to fully capture the essence of the consistency concept discussed in section 2, this priori highway length determination must be made very carefully in this research, and as a result,

2.4 m length was selected. This value usually covers 2 or 3 highway curves and tangent sections. Figure 3 presents how this research classifies the highway curve types.

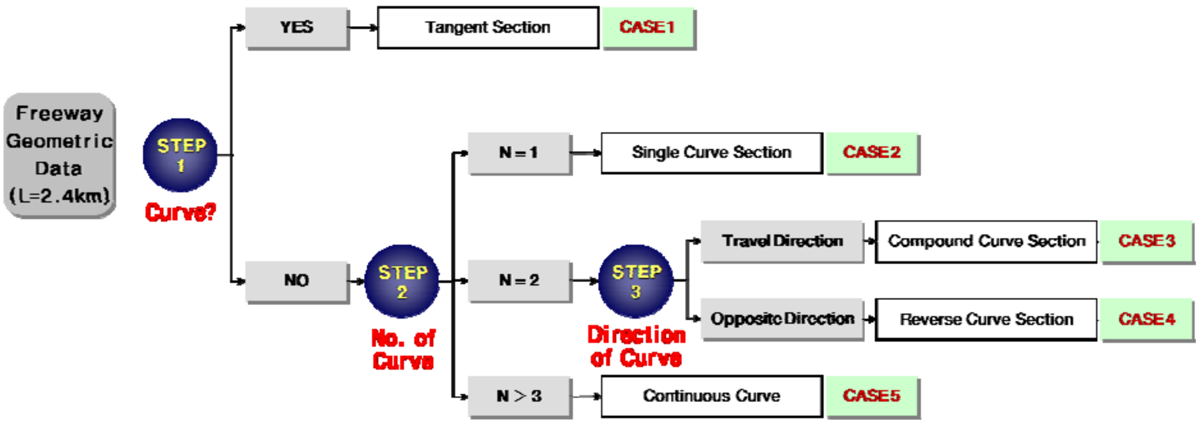



Figure 3 Classification of Highway Curve Types used in this Research

It is to be noted that few cases were observed for CASE 5, so this research precluded it in further analysis. Figure 4 illustrates the four cases of the highway curve type classification.

- CASE 1: Tangent Section
- CASE 2: Single Curve Section
- CASE 3: Compound Curve Section
- CASE 4: Reverse Curve Section

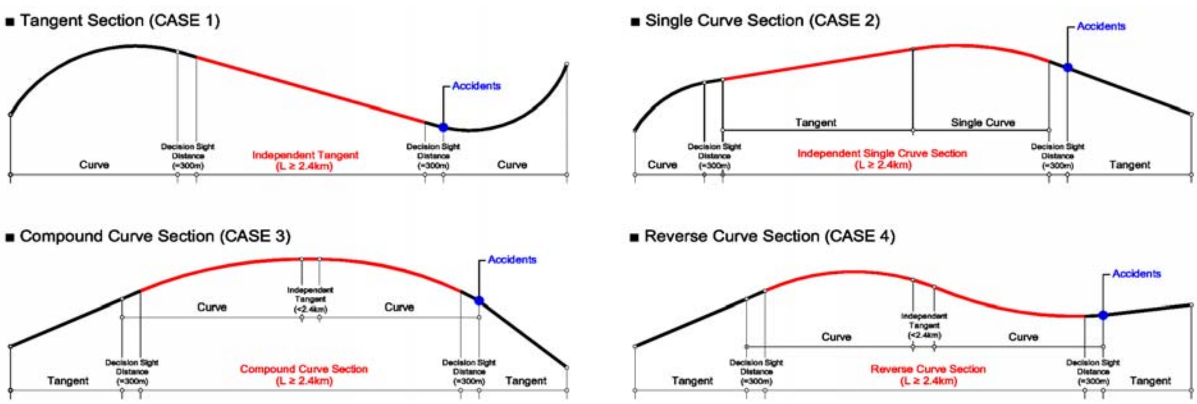



Figure 4 Four Cases of the Highway Curve Types

## 4.2 Regression Model

The multi-regression analysis was applied in this research to develop relationships between accident rate and highway geometric characteristics. This analysis assumes that dependent variables and independent variables are linearly related, and that dependent variables have equal variance regardless of the magnitudes of independent variables. However, it is accepted in crash model development that the variance exceeds the mean values. Therefore, variable transformation was required in this research. Eqn (1) is the regression model applied in this research.

$$Y = \beta_{0} + \beta_1 x_1 + \beta_2 x_2 + \cdots + \beta_n x_n + \varepsilon \quad (1)$$

| Variable Transformation Formula | Regression Model |
|---|---|
| Logarithm Model | $Y = \beta_{0} + \beta_1 \log(x_1) + \cdots + \varepsilon$ |
| Inverse Model | $Y = \beta_{0} + \frac{\beta_1}{x_1} + \cdots + \varepsilon$ |
| ⋮ | ⋮ |

Where, $Y$ : Dependant Variable

$X$ : Independent Variables

$\beta_1 \sim \beta_n$ : Coefficients

$\varepsilon$ : Error

### 4.3 Independent Variables

Table 4 shows the independent variables selected in this research that were expected to be able to explain the highway geometric characteristics influencing crashes on the road. Putting these independent variables into the four cases of highway curve types established previously, this research finally set a set of regression equations.

Table 4 Independent Variables used in this Research

| | Tangent Section | Curve Section |
|---|---|---|
| Dependant Variable | Accident Rate(100mil.veh.km) | |
| Independent Variable | Tangent Length(m),<br>Vertical Grade Value | Radius(m), Curve Length(m)<br>Vertical Grade Value |
| dummy Variable | Clothoid | |

### 4.4 Others

In the independent variable selection process, there were two things to mention. First, clothoid curves were also included in this analysis as dummy variable due to their contributions in providing drivers with better curve driving conditions. Second, in spite of apparent impact of the vertical grade on crash occurrences, it is challenging to include individual vertical grade values in the crash model development because these values usually keep changing on the road. Therefore, this research applied the average grade technique instead. Figure 5 and Eqn (2) present how this research treated the vertical grade.

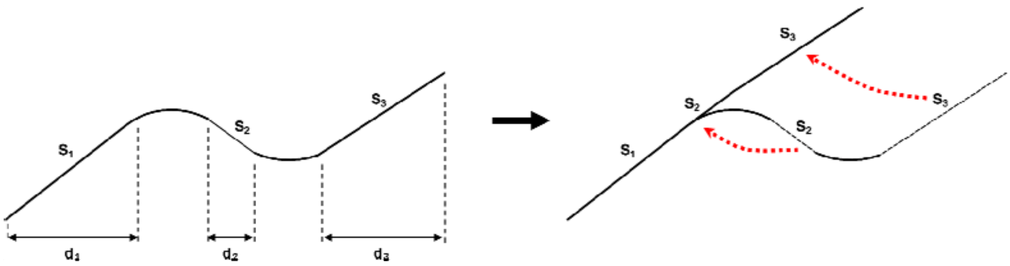



Figure 5 Average Vertical Grade Values used in this Research

$$Vertical\ \ Grade\ \ Value = \sum_{k=1}^{n} \left| \frac{S_k}{100} \times d_k \right| \tag{2}$$

### 4.5 Developed Models

Table 5 summarizes the developed models in this research. Based on these models, the followings can be stated:

- Longer than 2.4km tangent sections were found to be unsafe.
- Single curve Sections should be designed with longer radii and longer lengths to reduce crashes.
- Compound curve section and reverse curve section involved reduced crashes when two consecutive curves have little curve radius changes, and also showed crash reductions with longer tangent sections between them.
- Fewer crashes were observed when vertical grade values were small, and particularly so if the changes from adjacent sections were minimal.

Table 5 Developed Models in this Research

| | Developed Models |
|---|---|
| CASE 1 | $Y = 23.76\ln(L - DS \times A_1) + 1.57 Bend_{Ver.}$ |
| CASE 2 | $Y = \frac{3,127.612}{R} - 0.047\ln(CL - DS(A_1 + A_2)) - 0.02\ln(L) + 0.60 Bend_{Ver.}$ |
| CASE 3 | $Y = 0.003\lvert RR \rvert + 0.003\ln(CL - DS(A_1 + A_2)) - 0.03\ln(L) + 2.74 Bend_{Ver.}$ |
| CASE 4 | $Y = 0.001\lvert RR \rvert + 0.001\ln(CL - DS(A_1 + A_2)) - 0.03\ln(L) + 2.37 Bend_{Ver.}$ |

Where, $Y$ : Accident Rate(100mil.veh.km)

$L$ : Tangent Length (m)

$R$ : Radius (m)

$CL$ : Curve Length (m)

$DS$ : Decision Sight Distance (=300m)

$Bend_{Ver.}$ : Average Vertical Grades

$A_1$ : Dummy Variable

(0 : Existence of Clothoid in Beginning of Curve

1 : No Existence of Clothoid in Beginning of Curve)

$A_2$ : Dummy Variable

(0 : Existence of Clothoid in Ending of Curve

1 : No Existence of Clothoid in Ending of Curve)

### 4.6 Model Statistics

Table 6 summarizes the model statistics.

Table 6 Model Statistics

| | Confidence Level | R | $R^2$ | Adjust $R^2$ | Standard Error | t-Statistic | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | L | R | CL | RR | $Bend_{Ver.}$ |
| CASE 1 | 95% | 0.764 | 0.584 | 0.567 | 5.795 | 4.273 (0.000) | - | - | - | 2.632 (0.010) |
| CASE 2 | 95% | 0.811 | 0.657 | 0.636 | 5.626 | -6.728 (0.000) | 6.406 (0.000) | -3.952 (0.001) | - | 15.406 (0.000) |
| CASE 3 | 95% | 0.846 | 0.715 | 0.681 | 7.438 | -5.326 (0.000) | - | 5.723 (0.000) | 2.304 (0.015) | 5.662 (0.000) |
| CASE 4 | 95% | 0.838 | 0.703 | 0.675 | 7.245 | -2.740 (0.010) | - | 6.664 (0.000) | 5.906 (0.000) | 8.236 (0.000) |

It was found that the developed models based on classified highway curve types provided significantly higher correlation coefficients with 0.764~0.846. This indicates a high level of relation among included variables. Interestingly different from the normal cases of crash models that have very low determination coefficient values, this research provided models with 0.584~0.715, values rarely observed in crash analysis literature. Therefore, it is believed that this research approach was very effective in explaining crash occurrences for selected freeways. Table 7 is the analysis of variance for the developed models, and also supports the authors' conclusion.

Table 7 Analysis of Variances of Developed Models

| | | d.f | Sum of Squares | Mean Squares | F | Sig. F |
|---|---|---|---|---|---|---|
| CASE 1 | Regression | 2 | 1,771.83 | 885.92 | 33.69 | 0.000 |
| | Residual | 48 | 1,262.33 | 26.30 | | |
| | Total | 50 | 3,034.16 | | | |
| CASE 2 | Regression | 4 | 94,265.32 | 23,566.33 | 549.69 | 0.000 |
| | Residual | 1,148 | 49,216.67 | 42.87 | | |
| | Total | 1,152 | 143,481.99 | | | |
| CASE 3 | Regression | 4 | 4,362.48 | 1,090.62 | 32.65 | 0.000 |
| | Residual | 52 | 1,736.92 | 33.40 | | |
| | Total | 56 | 6,099.40 | | | |
| CASE 4 | Regression | 4 | 37,042.66 | 9,260.67 | 169.19 | 0.000 |
| | Residual | 286 | 15,654.03 | 54.73 | | |
| | Total | 290 | 52,696.69 | | | |

## 5. FINDINGS AND CONCLUSION

This research underscores the importance of applying the consistency concept in geometric design, and applied this concept to freeway crash analysis based on a set of highway curve patterns. The patterns were developed by capturing the crash occurrence characteristics of each different curve types. As a result, the followings were found in this research.

- Longer than 2.4km tangent sections were found to be unsafe.
- Single curve Sections should be designed with longer radii and longer lengths to reduce crashes.
- Compound curve section and reverse curve section involved reduced crashes when two consecutive curves have little curve radius changes, and also showed crash reductions with longer tangent sections between them.
- Fewer crashes were observed when vertical grade values were small, and particularly so if the changes from adjacent sections were minimal.

This research finding should be informative to engineers, and it is expected that this research approach would offer more accurate crash predictions than other existing approaches.

**REFERENCE**

AASHTO (2004), **A Policy on Geometric Design of Highways and Streets - 5th edition**, American Association of State Highway and Transportation Officials, Washington D.C

Bhagwant Persaud, Richard A. Retting, and Craig Lyon (2000), Guidelines for Identification of Hazardous Highway Curves, **Transportation Research Record 1717**, Transportation Research Board, Washington D.C.

Ezra Hauer(1999), Safety and the Choice of Degree of Curve, **Transportation Research Record 1665**, Transportation Research Board, Washington D.C.

Glennon JC (1987), Effects of Alignment on Highway Safety**, State of the Art Report Number 6: Relationship Between Safety and Key Highway Features**, Transportation Research Board, Washington D.C.

Kang J.K., Lee S.K.(2002), Traffic Accident Prediction Model by Freeway Geometric Types, **Journal of Korean Society of Transportation Vol.20 No.4**, Korea Society of Transportation, Seoul, Korea

Kenneth L. Fink, Raymond A Krammes (1995), Tangent length and Sight distance effects of Accident Rates at Horizontal Curves of Rural Two-Lane Highways, **The 74th annual meeting Transportation Research Board**, Washington D.C.

Kim K.S., Jin K.S.(1996), A Study on Interactive Relationship between Highway Geometric Elements and Driver Safety, **Korea Research Institute for Human Settlements**, Gyeonggi-do, Korea.

Lamm R.(1995), Safety Module for Highways Geometric Design, **Transportation Research Record 1512**, Transportation Research Board, Washington D.C.

McBean, P.A.(1982), The influence of road geometry at a sample of accident sites, **Laboratory Report LR 1053**, Transport and Road Research Laboratory, Crowthorne, UK

Raff M.S.(1953), Interstate Highway-Accident study, **Highway Research Board Bulletin 74**, Traffic Accident Studies Washington D.C.

Road Traffic Safety Authority (2002), **Master Plan of Black-spot Improvements and Measures of Effectiveness**, Road Traffic Safety Authority, Seoul, Korea

Zegeer C. V., Stewart R., Reinfurt D., Council F., Neuman T., Hamilton E., Miller T., and Hunter W.(1991), **Cost Effective Geometric Improvements for Safety Upgrading of Horizontal Curves**, FHWA-RD-90-021, Federal Highway Administration, Washington D.C.

Zegeer C. V., J. R. Stewart, and M. C. Forrest (1992), Safety effect of geometric Improvements on horizontal Curves, **Transportation Research Record 1356**, Transportation Research Board, Washington D.C.

Jovanis P., Chang H. (1986), Modeling the Relationship of Accidents to Miles Traveled, **Transportation Research Record 1068**, Transportation Research Board, Washington D.C.

Lee J., F. Mannering (1999), **Analysis of Roadside Accident Frequency and Severity and Roadside Safety Management**, Washington State Department of Transportation, Olympia

# A Study of Support System for Traffic Operating Condition Audit on Expressways

**Lee, Seungjun*, Choi, Yoonhyuk**, Kim, Taeyoung***, Kang, Jeonggyu******

*Transportation Research Division*

*Expressway & Transportation Research Institute*

**Abstract**

This research has been studied to construct support system for systematic approach of traffic operating condition audit (TOCA) to solve the serious traffic congestion lasting for more than 10 years on freeways. The support system for TOCA includes four major parts, such as decision methodology for congestion management, dynamic traffic assignment technique, traffic analysis software, and congestion management handbook.

This research has been performed according to prearranged scheme and the contents are follows: review of existing congestion index and microscopic traffic simulation model, development of the overall traffic condition index, dynamic O/D estimation, system design of traffic analysis computer program, doing an example of traffic condition audit, and so forth. In recent periods, ex (Ex (Korea Expressway Corporation)) has been under strong pressure to reduce traffic congestion on expressways. Therefore, this project focused on development of support system to help decision maker's policy-making and intended to play the role as a useful tool in future congestion management.

## 1. INTRODUCTION

### (1) Background

Expressway has been base of rapid economic growth of our country, on the other hand congestion on expressway has been severe and enlarged because of increase of traffic demand in recent. In 2007, the length of congested section on expressway was 292km, which was 9.3% of total length of 3,132km. Enlargement of capacity such like lane increasement and road construction has been used to remove congestion. However, to enlarge capacity requires long term period because planning, design and construction process is needed. Thus long lasting of congestion situation is unavoidable after congestion occurrence. Therefore, establishment of quick congestion management alternatives is required to improve existing congestion condition and to prevent added congestion.

This is a follow-up study to a series of studies on the development of highway traffic condition evaluation methods. This study aims to demonstrate the need to introduce a traffic flow evaluation system in order to manage traffic jams systematically and to develop ways to solve related problems. To that end, this study specifies the concept of a traffic condition evaluation system and develops various methods and ways to do so, and finally, systemizes those methods for adoption in real terms. Hence, this study aims at developing a specific, systematic, and fast way to develop a traffic condition evaluation system in order to manage possible traffic jams on highways.

### (2) Objective

The objective of this research is to make the concept of Traffic Operating Condition Audit (TOCA) more concrete, and to develop various techniques and methodologies to support TOCA. That is to say, the objective of this research can be said to develop support system for TOCA to manage congestion on expressway.

## 2. Methodology

In this research, an index for monitoring of traffic operating condition is developed and a methodology to analyze traffic condition is developed. Also, a decision-making methodology which enables decision makers to choose right traffic management alternative on each congestion condition is developed and traffic analysis programme to evaluate the effect of each alternative is developed. In addition, congestion management handbook including definition of congestion, congestion measuring method, classification of congestion, congestion management technique on each congestion section, method to find a robust alternative, and congestion management process is provided.

## 3. TRAFFIC OPERATING CONDITIONS AUDIT

Traffic Operation Condition Audit (TOCA) is defined as the process of comparing the many alternatives and methods on which to focus in order to understand the reasons for traffic jams and the methods of easing them. This evaluation process of TOCA is categorized into two systems; the prior evaluation to deal with possible traffic jams and the diagnosis of the reasons as to why these traffic jams occur.

In order to carry out a traffic conditions evaluation, we should select the range of the area to be evaluated and should collect the track records for the previous traffic conditions in that area. After collecting the required data, we should conduct studies on the ground while analyzing previously-collected data. Road geometric design, the current traffic conditions, the type of the traffic jams, the points where traffic jams occur, how long the traffic jams continue, and other questions are included in the research and analyses. After those processes, we should estimate the future traffic volume of the selected area in order to build up traffic management methods to prevent possible traffic jams before they occur. To that end, we select the areas that are currently suffering from, or will be suffering from, traffic jams. At the same time, we figure out the reason for those traffic jams and the influences they exert. In terms of the road geometric design and the characteristics of the traffic conditions, we find out the root cause of the traffic jam and analyze the traffic conditions of the road. In order to establish methods of preventing traffic jams from the start and in order to manage traffic jams that have occurred on the highway, we bring out different measures to solve the problems caused by the excessive demand of the road, decreased road capacity due to traffic conflicts, and the failure of the road management system. As a result, we set up measures to secure an appropriate road capacity by preventing traffic conflicts and managing the traffic system more effectively while conducting an evaluation of the management systems of each local government.

## 4. SUPPORT SYSTEM FOR TOCA

In order to evaluate traffic conditions, we should figure out the root cause of traffic jams, find out the solutions, compare the alternatives, and evaluate the effectiveness of the adopted measures. In each step, we should develop tailored methods and connect them with each other. However, previous studies have only focused on developing and introducing alternatives to ease traffic jams. Thus, we have failed to figure out the exact cause of the traffic jams and to monitor how traffic jams change. At the same time, we have failed to establish a reasonable decision-making process to select the best alternatives based on the priorities, the feasibility, and the effectiveness of each policy and alternative. In order to support the evaluation of traffic conditions in a faster, more accurate and effective manner, this study sets up 4 objectives, which are the key elements of the traffic conditions evaluation support system. Four major contents are as follows;

First, traffic operating condition monitoring index

Second, decision-making methodology for congestion management

Third, traffic analysis program

Fourth, congestion management handbook

## 5. DEVELOPMENT OF A SUPPORT SYSTEM FOR TOCA

### (1) Develop a traffic conditions monitoring indicator

Generally, congestion means traffic jams or traffic delays caused by excessive road traffic demand, problems of road design, and car accidents and others. Hence, the time it takes to get to somewhere during a traffic jam or traffic delay minus of the tie it takes during freely flowing traffic is the congestion time.

The existing traffic congestion index is not based on comprehensive data representing all traffic conditions, but is based upon independent data, such as traffic volume, velocity, time, and others. Even though cars run at the same speed, the severity of the traffic congestion and its length of time can vary, but these kinds of aspects are not well reflected in the previous traffic congestion index. In addition, the existing traffic congestion index is only focused on the current status of the road and there is no standard by which to judge which level of traffic flow is appropriate. Therefore it is hard to set a norm defining the appropriate level of traffic flow.

Thus, this study wants to develop a comprehensive traffic indicator which shows the traffic flow level objectively.

### (2) Development of a comprehensive traffic conditions index

The Traffic Conditions Index is defined as an index which measures the traffic circumstances and evaluates them comprehensively. Also, it is the index that shows the traffic status of a certain area objectively. Unlike existing traffic congestion indicators, the TCI focuses on 3 qualities of service; the

mobility (how long does it takes to get somewhere?), the accessibility (how easily can you get somewhere?), and the punctuality (can you get to somewhere in time?).

The TCI aims at expressing an exact traffic condition objectively based on the traffic congestion management goal, which is the standard defining traffic congestion. Therefore, it is important to set the standard or the traffic congestion management goal first. Generally, in the Highway Capacity Manual, LOS E means the peak capacity of the road; however the designated speed or designated service level of the road is used as a standard to define the level of highway operation. This study sets the graph with car speed and the designated service level as a standard with which to manage a traffic congestion and compares the traffic conditions as well as the characteristics of LOS C, D, and E. Finally, it develops a TCI based on LOS D as a standard by which to manage traffic congestion.

**(3) Development of the 1st model**

TCI takes the density of the road as the main variable to reflect mobility and accessibility simultaneously. The TCI index is based on the difference between the designated density of the road and the traffic density.

$$1stTCI = \frac{DesignDensity}{MeasuredDensity}$$

However, traffic density is hard to calculate arithmetically, so we calculate the traffic density of a certain area based on a formula that includes traffic volume, velocity, and traffic density.

**(4) Development of the 2nd model**

**A more correct and realistic traffic conditions index**

When we divide the designated density against the traffic density to define the traffic conditions, we cannot fully consider an area which is not sensitive to a change of traffic speed or which is already under traffic congestion conditions. In order to resolve this problem, we take the 'traveling vehicle speed' into consideration when calculating the traffic conditions. In addition, we use the concept of 'designated speed' to compare the TCI of a certain area with that of other areas. By doing so, we can show the traffic congestion conditions more accurately and compare the TCI of one area with another area.

**The division of traffic conditions**

In the first model, the range of the index showing smooth traffic conditions is very wide, but that of normal traffic conditions and traffic congestion is too narrow to be changed sensitively according to the road conditions. This means that the traffic conditions index is not sensitive to a change in the road conditions, so the drivers and managers of the traffic systems are not able to be aware of slight changes in the traffic conditions. Therefore, in the second model, we have adjusted the range of the traffic conditions index in each category based on the real traffic conditions.

These characteristics of TCI show that TCI is quite different from other existing indices. Table 1 shows the equation of TCI calculation and the criteria for traffic operating condition. As shown in Table 1, traffic operating condition is divided as three categories of good, fair and poor.

Table 1. Criteria for traffic operating condition

| Condition | Criteria | TCI |
|---|---|---|
| Good | 70≤ TCI ≤100 | $70+(x-920)\times\frac{30}{(\max-920)}$ |
| Fair | 50≤ TCI <70 | $50+(x-543)\times\frac{20}{(920-543)}$ |
| Poor | 0< TCI <50 | $x\times\frac{50}{(543-0)}$ |

$$x=\frac{DesignDensity}{MeasuredDensity}\times\frac{MeasuredSpeed}{DesignSpeed}\times 1000$$

**Feasibility study**

In order to carry out a feasibility study on the TCI, we collected VDS data from the area between Osan IC and Seoul TG on the Gyeongbu highway and the area between Bibong IC and Jonan JC (387.1 ~ 403.3 km (25.2 km)) from October 15, 2007 to October 21, 2007. The n we were able to calculate the TCI and analyze the exact traffic conditions.

Figure 1 shows the comparison between TCI and other indices (volume, speed, TTI) for traffic operating condition on Kyeongbu Expressway (Osan - Seoul). In Figure 1, TCI presents the traffic operating condition as three categories with different other indices. Thus, it might be possible to guess that TCI is more realistic than other indices.

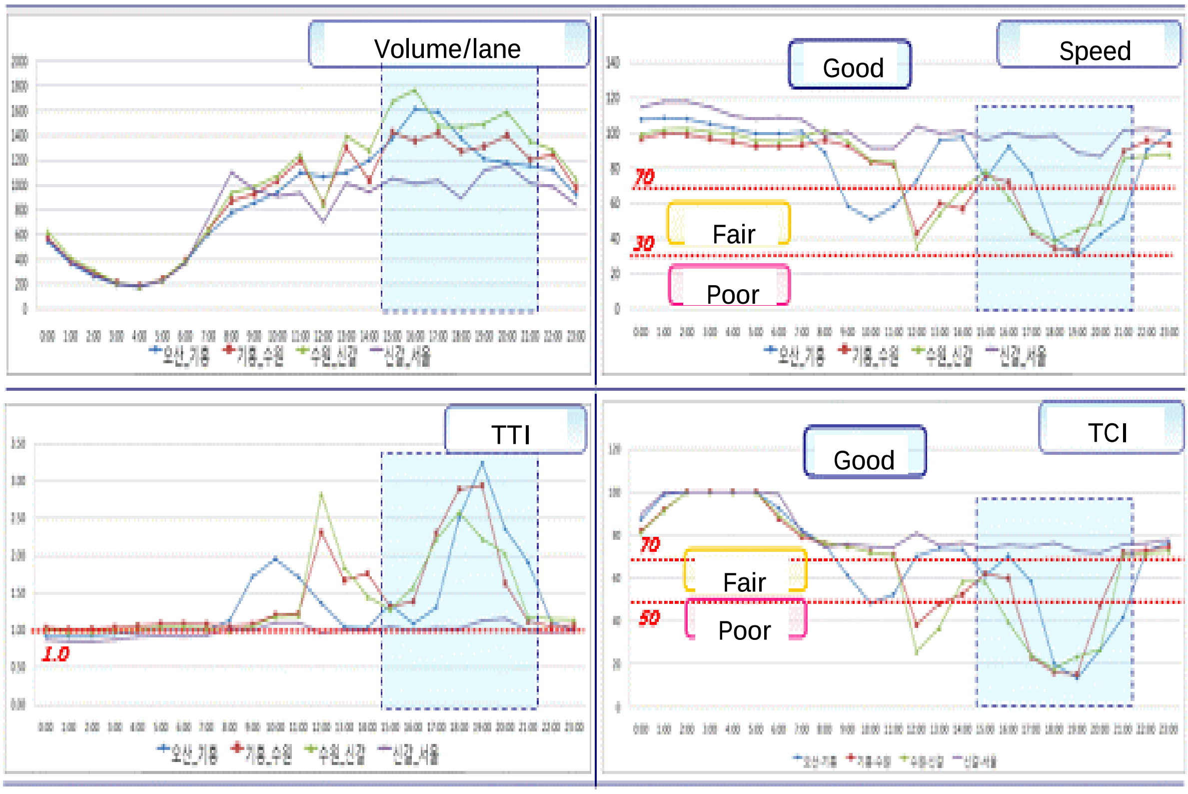



Figure 1. Comparison between TCI and other indices

According to the results, the TTI only showed the level of traffic congestion based on the current vehicle travel speed compared to the freely flowing speed. Thus it failed to reflect the real traffic conditions exactly because there is no value standard with which to divide smooth, normal, and heavy traffic conditions. In addition, it defines traffic congestion when the vehicle travel speed is slower than 30 km/h, while the speed between 30 to 50 km/h is not considered to be in the status of traffic congestion. Thus, it failed to reflect reality accurately.

However, the TCI can show the exact traffic conditions more clearly than the TTI because it has a standard by which to define traffic congestion and traffic service quality. For example, an area between Suwon and Shingal was considered to be a congested area in the previous studies. The TCI showed a similar result to reality but the value of the TTI failed to reflect this.

## 6. Decision-Making Methodology for Congestion Management

Evaluation on the effect of alternatives is important to select a robust alternative among various congestion management alternatives. Additionally, development of decision-making methodology considering the feasibility and the priority of policy is required. In this research, a new methodology integrating existing measurable and unmeasurable method was developed.

This research reinforced existing measurable method through adaptation of traffic volume and vehicle ratio to calculate more accurate benefit. Moreover, in this research, classification of short-term and long-term alternatives was established. On the other hand, in unmeasurable method, AHP technique was adapted to analyze the priority, feasibility and necessity of policy.

Final decision-making for congestion management is determined by considering the results of both methods

Decision-making process for congestion management is composed of monitoring, decision-making and historic data saving stage.

### 7. Traffic Analysis Programme

Traffic analysis programme is comprised of data input module (road geometric condition, traffic demand, and so forth), traffic analysis module, traffic demand estimation module, decision-making module. Figure 2 shows the concept of traffic analysis programme.

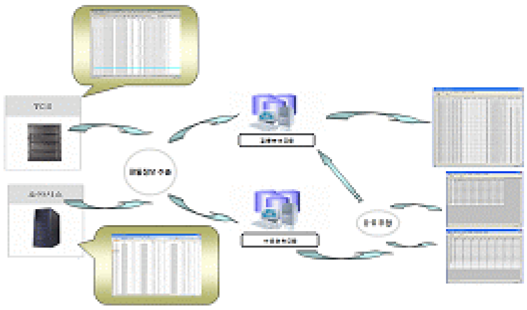


Figure 2. Concept of traffic analysis programme

As shown in Figure 2, raw data is obtained from TCS and FTMS and effective data such like O/D, traffic volume in each time interval (slice) as input data to analysis is derived from algorithm developed in this research. In Figure 3, the analysis flow of traffic analysis programme is presented.

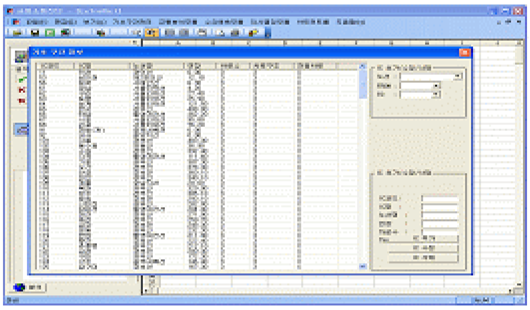


Figure 3. Example of data input

Figure 4 is the example of congestion analysis result. In Figure 4, it is possible to know that where the queue occur and to where the effect of congestion reach. And it is also possible to see that when the congestion happens and disappear. In addition, traffic analysis programme has the function to compare the results between both alternatives.

Figure 5 shows the summarized report. In summarized report, comparison of effect between both alternatives is provided.

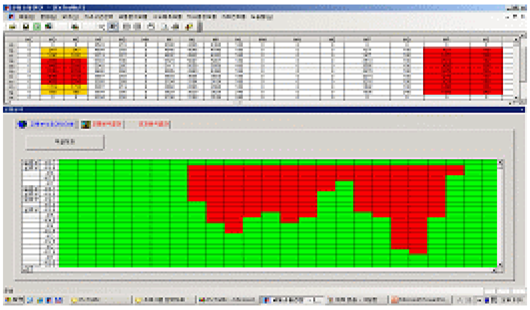


Figure 4. Congestion analysis results in time-space diagram

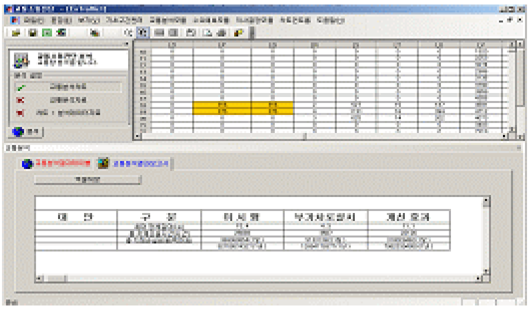


Figure 5. Summary - comparison of effect between both alternatives

## 8. Congestion Management Handbook

In this research, congestion management handbook is provided. The goal of the handbook is to give some guidelines to traffic managers and thereby, it might be expected that to perform successful traffic management is easy and possible.

Figure 6. Congestion management handbook

**9. Conclusions and future studies**

This study was carried out to establish a support system for traffic conditions evaluation. We have spent about 10 years researching how to define and deal with traffic congestion. Notably, as the Ex (Korea Expressway Corporation) tries to ease traffic congestions on the highways, we want to establish scientific and systematic traffic condition management methods while providing valuable data for working groups and executives to make the appropriate traffic congestion control policies. Therefore, the TCI and traffic congestion management methods are expected to help us to establish traffic congestion management plans by showing changes in traffic conditions on the road and diagnosing the root cause of traffic congestions. In addition, the traffic analysis electronics program will be helpful to anticipate the influence of traffic congestion and future changes in traffic conditions. Also, the program will help us to evaluate many alternatives and find out the best solution to ease traffic congestion. At the same time, it can also be used as basic data for policy development, including the construction, expansion, modification, and improvement of the highways. Lastly, the working level officials in the highway management department can use these data to establish traffic congestion management policies.

This study created an electronic system to evaluate traffic conditions based on the traffic data. However, further studies and system integration efforts are needed to establish real-time traffic conditions evaluation systems in cooperation with the FTMS and Road Plus of the Ex (Korea Expressway Corporation). In addition, this is a basic study to analyze dynamic traffic conditions. Therefore, we need to conduct further studies to anticipate future dynamic traffic demands. In the long term, more traffic analysis models, tailored to the environment of each highway, are needed.

## 10. REFERENCE


1. California Department of Transportation Division of Research & Innovation, Transportation System Impacts Cross- cutting Evaluation Report, 2004.

2. CAMPO, “Congestion management system (CMS) state of the system report”, 2004.

3. Chao Chen, “Cause and Cures of Highway Congestion”, 2001.

4. FHWA, “Freeway Management and Operations Handbook”, 2003.

5. Pravin Varaiya, “How to measure Transportation system performance”, 1997.

6. Pravin Varaiya, “Freeway Performance Measurement System (PeMS), Version 3 Phase Ⅱ”, 2004.

7. Quantifying congestion, NCHRP Report 398 Vol 1, 1997.

8. Quantifying congestion: user's guide, NCHRP Report 398 Vol 2, 1997.

9. Texas Transportation Institute, “Monitoring Urban Roadway in 2001; Examining Reliability and Mobility with Archived Data”, 2003.

10. Texas Transportation Institute, “Monitoring Urban Freeways in 2003 : Current Conditions and Trends from Archived Operations Data”, 2004.

11. Texas Transportation Institute, “2004 Urban Mobility Report”, 2004.

12. Texas Transportation Institute, “2005 Urban Mobility Report”, 2005.

13. The PeMS algorism for accurate, real-time estimates of g-factors and speed from single-loop detectors, IEEE ITS Conference Proceedings, 2001.

14. Transportation Research Board, “FREEWAY PERFORMANCE MEASUREMENT SYSTEM : MINING LOOP DETECTOR DATA”, 2001.

# Monitoring and Analysis of Weekday Bus-lane policy on Gyeongbu Expressway

**Lee, Seungjun*, Choi, Yoonhyuk**, Kim, Taeyoung*****

*Transportation Research Division*

*Expressway & Transportation Research Institute*

## Ⅰ. Introduction

### 1. Background and objectives

The traffic volume of South Korea has risen rapidly since 1990, but road capacity failed to meet those skyrocketing demand. Thus, we are witnessing frequent traffic delays and traffic jams. Currently, the traffic volume of the highway connecting the Seoul metropolitan area with Gyeonggi province is rising annually due to the development of new residential districts and cities. However, the traffic volume of the highway connecting Seoul and Busan, so called Gyeongbu Expressway, has risen too much to call it as a highway. According to the data showing the annual traffic volume of Gyeonggi province, we can see that the traffic volume has been skyrocketed since it was 121.349(Unit : a thousand cars) in 1992.

The ministry of land, transport, and maritime designated a buy-only lane in the 44.8Km-long section from the southern part of Hannam bridge of Han River to Osan IC and has operated the bus-only lane system since July, 2008. This system aims at vitalizing public transportation usage as well as decreasing the demand for cars to lay a firm foundation for green growth and to reduce social costs. However, there is no specific study or data focusing on the effectiveness of this system. Therefore, this study compares and analyzes the influence of the Weekday Bus-lane policy on the Gyeongbu expressway objectively. At the same time, it monitors the traffic volume and traffic condition of the expressway, so that the many related organizations can use these data to establish related policies in the future.

### 2. Methods

This study reviewed related papers on the bus-only lane policy and monitored the change of the traffic condition after implementing the policy. It compared the traffic volume at Seoul TG, East Seoul TG, and West Seoul TG in 2007 and 2008. To identify how the traffic pattern changed after the implementation of the bus-only lane policy, it compared the rate of monthly traffic volume change, the traffic volume of passenger cars, and the number of bus passengers in the first half of 2007 and 2008 with that in the second half of 2007 and 2008. Also, traffic condition was monitored through CCTV data collected in the Gyeongbu expressway. Based on the comprehensive data collected by Korea Expressway Corporation, the rate of traffic jams and traffic delays in the section with bus-only lanes were calculated.

## Ⅱ. Review of previous studies

### 1. The definition of a bus-only lane in Gyeongbu expressway

The bus-only lane of Gyeongbu expressway is similar to HOV (High occupancy vehicle) lanes of foreign countries. Cars with a capacity of 9people and light-duty vehicles are also allowed to use the bus-only lane. Construction and designation of bus-only lanes need relatively short period of time and less money than other methods to ease traffic jams. Also the bus-only lane policy encourages people to use public transportation more. As a result, we can use roads more efficiently as well as ease severe traffic jams. At the same time, we can shorten the driving time and reduce driving costs with this system, so that total socio-economic cost of transportation can be cut.

The Road Traffic Act is the legal background of the designation of bus-only lanes in Korea. According to the 61st Article of the Road Traffic Act, Commissioner General can designate an exclusive lane including bus-only lanes in expressways when it is needed to maintain smooth traffic flow in expressways. Other categories of possible exclusive lanes are listed in the executive order of the Road Traffic Act.

### 2. History of the bus-only lane of Gyeongbu expressway

Bus-only lanes in the Gyeongbu expressway have been temporarily operated in a section between Yangjae and Shintanjin since July, 1994 to ease severe traffic jams during holiday seasons. After that, Weekday bus-only lane policy was implemented in a section between Suwon IC and Seocho IC as a pilot project in September, 2003 for a month. The lane was also allowed to car with 9-passenger capacity when there were more than 6people in the car. During morning rush hours, the bus-only lane was operated from 7 to 9 to Seoul and during evening rush hours, the lane was operated from 6 to 8 to Busan. However, after expanding the road connecting Hannam Bridge with Banpo IC, the bus-only lane was abolished due to the need for more feasibility studies. From the July 1, 2008, the ministry of land, transport, and maritime, the Seoul metropolitan government and the police agency temporarily implemented a weekday bus-only lane policy for 3month in an section between Osan and Hannam bridge to encourage people to use public transportation more when oil prices are going higher and higher and to shorten the commuting time for citizens living in the southern part of Seoul. From October, 2008, the system was officially implemented and the operating time of the system is from 7 to 9 in the morning.

## Ⅲ. Monitoring Analysis

### 1. Changes of the traffic volume

#### 1) Comparison of traffic volume change in terms of time and location

This study compared the traffic volume change in the Seoul TG according to the time to find out how much the weekday bus-only lane policy affects traffic volumes. At the same time, to figure out the traffic volume of the road which Gyeongbu expressway users can detour, we also compared traffic volume change in the Seohaean expressway West Seoul TG and Jungbu highway East Seoul TG. To analyze them based on time, we also used daily average traffic volume of those roads and the monthly average of traffic volume is shown in the table 1 and picture 1.

Table 1. Average monthly volume of 2007 and 2008 (unit : veh/day)

| | Seoul TG | | West Seoul TG | | East Seoul TG | |
|---|---|---|---|---|---|---|
| | 2007 | 2008 | 2007 | 2008 | 2007 | 2008 |
| Jan | 200,465 | 200,310 | 156,696 | 162,022 | 112,962 | 114,032 |
| Feb | 205,856 | 208,281 | 166,085 | 171,959 | 126,278 | 122,606 |
| Mar | 210,593 | 214,688 | 164,616 | 174,581 | 120,400 | 122,436 |
| Apr | 217,976 | 217,434 | 171,354 | 179,146 | 128,994 | 129,690 |
| May | 218,089 | 218,432 | 171,892 | 180,130 | 133,238 | 131,436 |
| Jun | 214,295 | 210,608 | 172,671 | 175,672 | 130,649 | 122,851 |
| Jul | 212,699 | 196,517 | 170,376 | 175,782 | 129,274 | 125,371 |
| Aug | 215,738 | 203,169 | 173,669 | 185,231 | 139,533 | 140,189 |
| Sep | 218,058 | 203,131 | 174,814 | 183,474 | 134,055 | 132,838 |
| Oct | 221,016 | 194,810 | 176,403 | 186,040 | 130,433 | 132,860 |
| Nov | 218,907 | 191,633 | 175,365 | 185,680 | 131,177 | 132,490 |
| Dec | 208,615 | 183,674 | 167,579 | 175,572 | 123,395 | 119,767 |
| Total | 2,562,307 | 2,251,054 | 2,041,520 | 1,949,609 | 1,540,388 | 1,394,076 |
| Ave. | 213,526 | 204,641 | 170,127 | 177,237 | 128,366 | 126,734 |

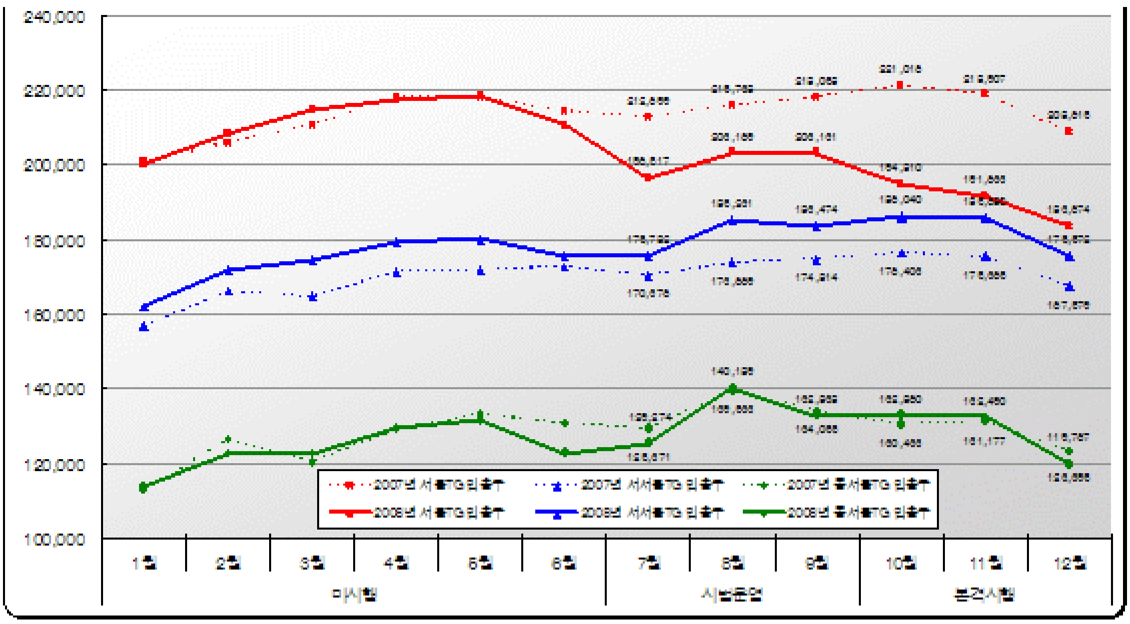



Figure 1. Comparison of average monthly volume from 2007 to 2008

According to the comparison of daily average traffic volume of each month in 2007 and 2008, the traffic volume change of the expressway was notable in the Seoul TG, followed by West Seoul TG, and East Seoul TG. In the case of Seoul TG, the traffic volume was all around the same from 2007 to May, 2008 but from June, the traffic volume of it started to decrease. In the case of West Seoul TG, the average traffic volume of

2008 increased compared to that of 2007 and in the case of East Seoul TG, the traffic volume of 2 years was similar except June, 2008.

What grabs our attention is the drop of the traffic volume in June, 2008. The main reason behind it was skyrocketing oil prices when we start to implement weekday bus-only lane policy. Due to high oil prices, the traffic volume of Seoul TG, and that of East Seoul TG in June, 2008, decreased from the previous year. The traffic volume of West Seoul TG was also decreased but still higher than that of June, 2007. After August, 2008, the traffic volume of West Seoul TG increased from the previous year but that of Seoul TG decreased due to the implementation of Weekday bus-only lane policy. After October, 2008, when the weekday bus-only lane system was officially implemented, the traffic volume rapidly decreased. However, the traffic volume of the same section increased in August and September due to the summer vacation season but was still lower than that of previous year.

According to the analysis, the traffic volume of Gyeongbu expressway was proved to be decreased after the implementation of Weekday bus-only lane policy. However, it is hard to say that people used public transportation more or many drivers detoured to another roads. However, given that the traffic volume also decreased by high oil prices, some amount of traffic volume of Seohaean Expressway or Jungbu Expressway might be transferred to other roads.

### 2) Analysis of traffic flow pattern

We took a look at the change of traffic volume at the TG of each expressway but it was hard to make a clear view on the change of traffic volume after conducting Weekday bus-only lane policy in the Gyeongbu expressway. Therefore, based on the average traffic volume from January to June of 2008, before the implementation of Weekday bus-only lane policy, we compared the monthly average traffic volume of 2007 and 2008. We set the traffic volume of the first half of 2007 and 2008 as a standard because Weekday bus-only lane system was implemented from July 2008. The result of analysis is in the table 2 and picture 2.

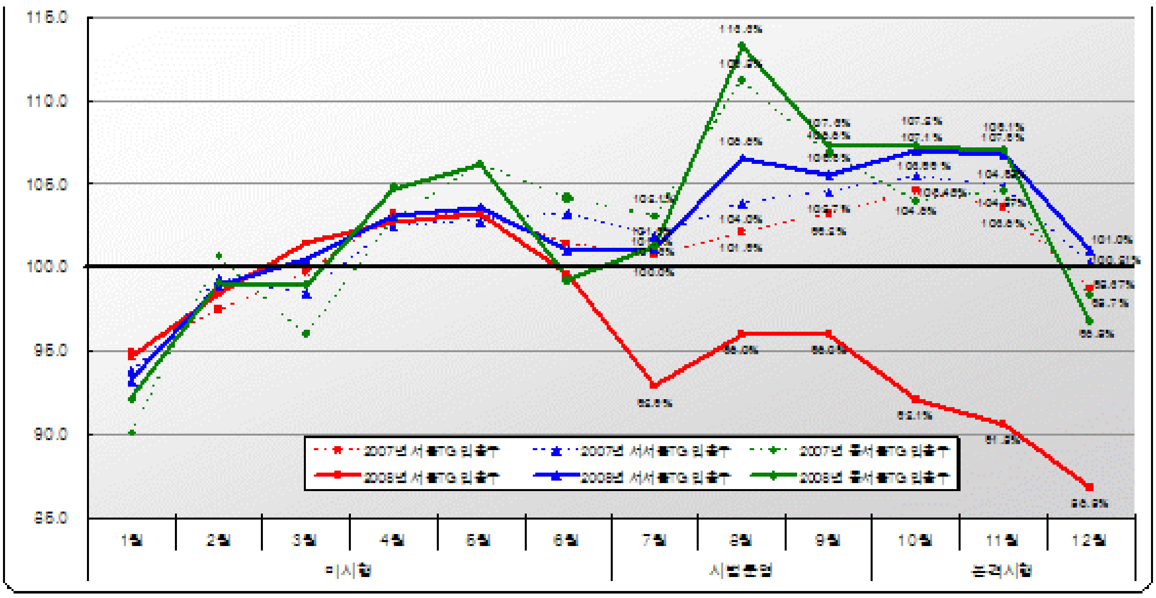



Figure 2. Monthly rate of average traffic volume based on the average volume of first half year

Table 2. Monthly rate of average traffic volume based on the average volume of first half year

| | | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec | Ave. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Seoul TG | 2007 | 94.9 | 97.4 | 99.7 | 103.2 | 103.2 | 101.4 | 100.7 | 102.1 | 103.2 | 104.6 | 103.6 | 98.7 | 101.1 |
| | 2008 | 94.7 | 98.4 | 101.5 | 102.7 | 103.2 | 99.5 | 92.9 | 96.0 | 96.0 | 92.1 | 90.6 | 86.8 | 96.2 |
| West Seoul TG | 2007 | 93.7 | 99.3 | 98.4 | 102.5 | 102.8 | 103.3 | 101.9 | 103.9 | 104.5 | 105.5 | 104.9 | 100.2 | 101.7 |
| | 2008 | 93.2 | 98.9 | 100.4 | 103.0 | 103.6 | 101.0 | 101.1 | 106.5 | 105.5 | 107.0 | 106.8 | 101.0 | 102.3 |
| East Seoul TG | 2007 | 90.1 | 100.7 | 96.0 | 102.8 | 106.2 | 104.2 | 103.1 | 111.2 | 106.9 | 104.0 | 104.6 | 98.4 | 102.4 |
| | 2008 | 92.1 | 99.0 | 98.9 | 104.8 | 106.2 | 99.2 | 101.3 | 113.3 | 107.3 | 107.3 | 107.0 | 96.8 | 102.8 |

According to the analysis, the monthly average traffic volume of Seohaean Expressway and Jungbu Expressway were at the similar level but that of Gyeongbu Expressway was decreased rapidly.

In the case of Seohaean Expressway and Jungbu Expressway, the traffic volume of the second half of 2008 increased slightly but in the case of Gyeongbu Expressway, the volume decreased by 7.8% compared to the previous year and we can say that Weekday bus-only lane affected to this change. However, the volume increased in August and September slightly due to the summer holiday season. In addition, given that the system was implemented as a pilot project of bus-only lanes from July to September in 2008, there might be some violation of the system and it might contribute to the increased traffic volume in August and September in 2008.

From October, 2008, when the system was officially implemented, the traffic volume of Jungbu Expressway and Seohaean Expressway increased by 3.3% and 1.5% respectively compared to the previous year but that of Gyeongbu Expressway rapidly decreased by 12.5%. What is notable is that the traffic volume at TGs of Jungbu Expressway and Seohaean Expressway was higher than that of the previous year by 2.1% and 2.6% respectively in August. The main reason of this phenomenon is increased traffic volume due to the summer holiday season.

### 3) Change of passenger car volumes and the number of bus passengers

Analyses on the traffic volume and the traffic pattern was based on the total cars so that we needed to figure out how many passengers started to use buses connecting Gyeongi province and Seoul after the implementation of Weekday bus-only lane policy. According to the analysis on the bus passenger numbers, the number of bus passengers has increased continuously and the driving speed of bus also increased. The number of bus passengers increased by 30.6% (58570 persons/day) on average after conducting the policy and this shows that many started to use public transportation after the implementation of the policy. There may be 3 reasons behind it.

First, the discount policy for bus transfer was expanded to red buses, so that the bus fare was dropped. Second, the red bus lines were reformed hugely, resulting in an enhanced quality of public transportation system. In fact, the number of bus lines increased from 29lines to 46lines after the reform. Finally, after the implementation of Weekday bus-only lane policy, the driving speed of bus increased and accuracy of buses was also enhanced. As a result, the quality of public transportation service was improved as a whole.

In addition, we compared the number of passenger cars on the road. To do so, based on the figures in the Seoul TG of Gyeongbu Expressway, we compared the number of passenger cars and all kinds of cars of each month from January to December of 2007 and 2008. We used TCS data collected by Korea Expressway Corporation.

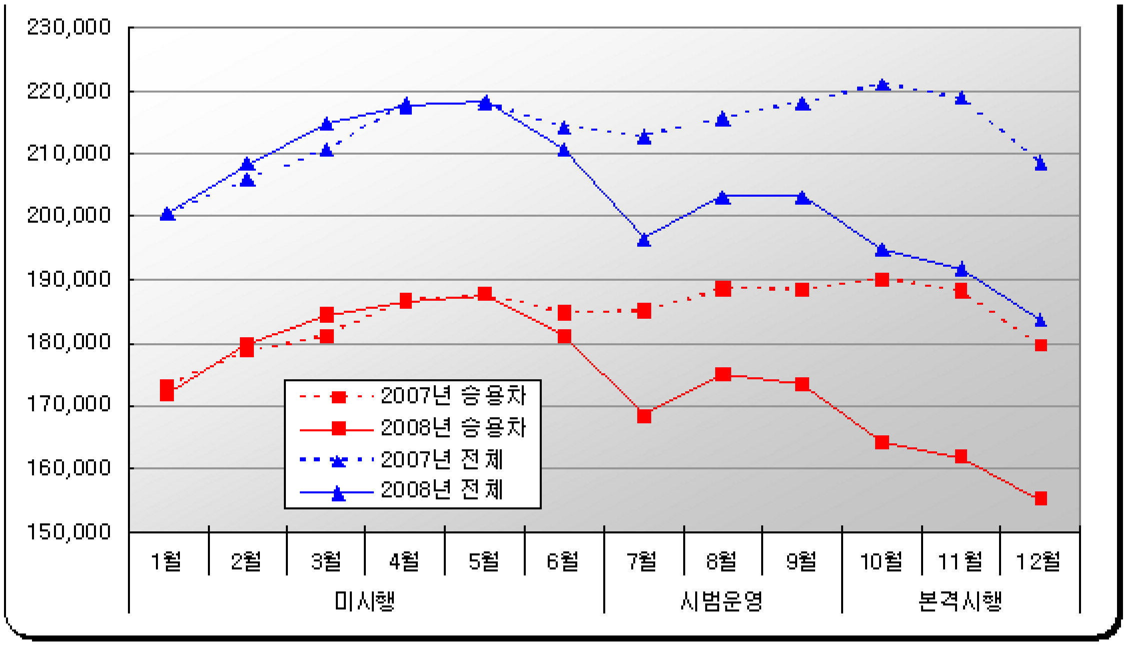



Figure 3. Number of passenger cars in the Seoul TG of Gyeongbu Expressway

According to the comparison, there was no difference between 2007 and 2008 in terms of the number of passenger cars and all kinds of cars before the implementation of Weekday bus-only lane policy but during the pilot project period, the number of passenger cars decreased by 8% compared to that of 2007 and when we started to conduct the policy officially, the number of passenger cars decreased by 13.72% and the total traffic volume decreased by 12.09% compared to that of 2007.

What is notable is that the number of passenger cars decreased due to the implementation of Weekday bus-only lane policy and the range of that decrease was relatively huge after the official implementation of the policy. The main reason of this huge change was that there were no crackdown on the use of bus-only lanes during it was a pilot project. After October, 2008, when the crackdown on the bus-only lane usage started, other lanes were congested due to bus-only lanes. Thus, many drivers detoured to other local road as well as national highways. Also some drivers might decide to use public transportation to avoid those kinds of traffic congestion caused by the implementation of the bus-only lane policy.

## 2. Change of traffic congestion patterns

### 1) Comparison of traffic congestionrn based on the CCTV data

This study monitored traffic delays and traffic jams after the implementation of Weekday bus-only lane policy in the Gyeongbu expressway while analyzing traffic volume in the expressway. To that end, we used CCTV located along the road from Osan to Hannam bridge of Han River. According to the monitoring, the

traffic congestion took place to Giheung before the implementation of the policy, to Osan during the pilot project, and to Ansung after implementing of the system. These results showed that the traffic congestion in the expressway became more severe after the implementation of the bus-only lane system. In the side to Busan, the traffic congestion occurred in Suwon reached to Shingal IC before the implementation of Weekday bus-only lane policy but the congestion reached to the Seoul TG after it.

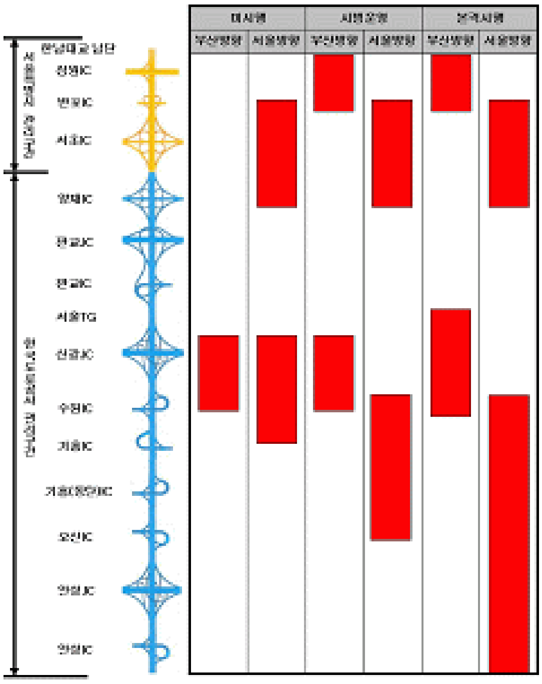


Figure 4. Change of traffic congestion patterns

According to the analysis, the traffic congestion in both sides of Gyeongbu expressway got severe much after the implementation of Weekday bus-only lane policy. In both sides, the congestion started to take place in Suwon. In the side to Seoul, the traffic congestion started at Suwon and reached to Osan and Ansung. In the side to Busan, the traffic congestion started at Suwon and reached to Shingal and to Seoul TG.

If we divide the bus-only lane implementation area into two sections; one is Seoul(Hannam- Yangjae) and the other is metropolitan area(Seoul TG- Ansung), the traffic condition of the Seoul area did not change much after conducting bus-only lane policy while that of the metropolitan area changed a lot. The main reason of this difference was that there was a bottleneck phenomenon in the metropolitan area.

Especially, when we take a look at the change of traffic congestion with the change of traffic volume, we can know that the traffic congestion got severe even though the total traffic volume decreased after the implementation of Weekday bus-only lane policy. It shows that even thought the traffic volume decreased due to the figure, traffic congestion got more severe because we lost a lane for all vehicles except buses. In conclusion, the fact that the traffic volume at the Seoul TG decreased but the traffic congestion got more severe shows us that the inconvenience occurred by this policy is much bigger than the merit of it.

### 2) Analysis on the traffic congestion rate based on the traffic congestion data

We analyzed traffic condition of each section of the expressway briefly based on the CCTV data but we needed to have some exact figures to clarify the level of traffic congestions. To that end, we used FTMS traffic congestion data collected by Korea Expressway Corporation to identify traffic condition of the expressway. We used the traffic congestion data collected from January 2008, to the 23rd, December, 2008, to figure out the change based on the average traffic congestion calculation methods per time or per section. After the implementation of the bus-only lane policy in the Gyeongbu expressway, the time to analyze traffic conditions was from 6 A.M. to 10 P.M.

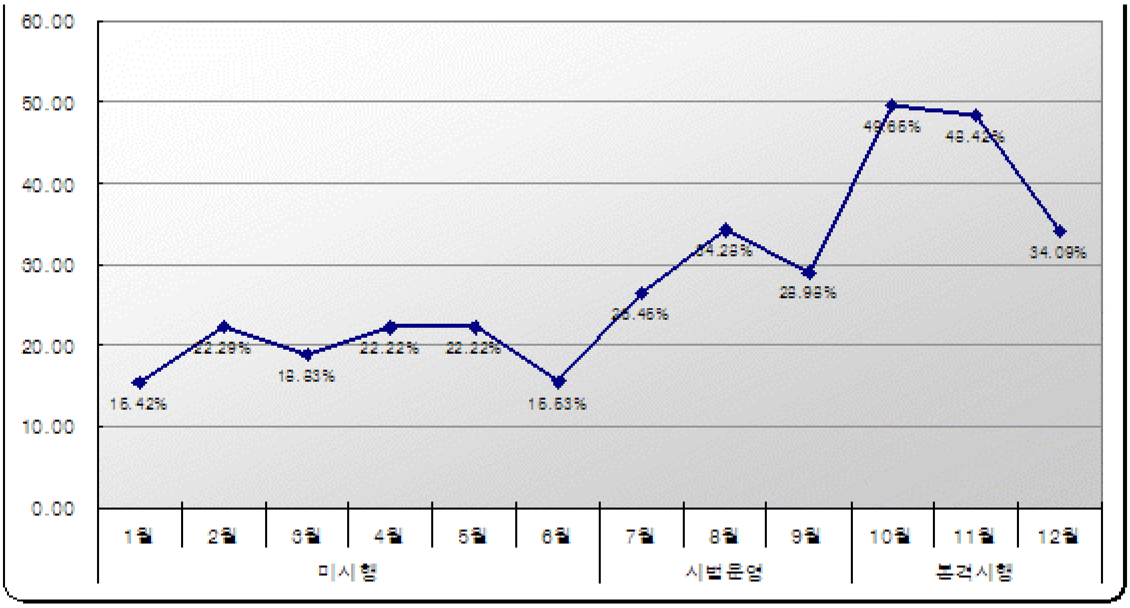



Figure 5. Change of traffic congestion rate

The traffic delay rate of Gyeongbu expressway was at 19.42% before implementing Weekday bus-only lane policy, but the rate increased to 29.9% during pilot project period, and increased to 44.1% after implementing the bus-only lane policy officially. This result showed that the traffic congestion of ordinary roadways got severe because of the car which couldn't use the bus-only lane. What is notable is that the traffic congestion rate has increased even though the traffic volume as a whole decreased.

Also, the traffic congestion rate of the expressway has decreased slightly since November, 2008. It may be because shoulder lanes are used in some sections of the Gyeongbu expressway to prevent possible traffic jams occurred by the implementation of bus-only lane policy. From Shingal IC to Jukjeon Highway service area, the driving speed of the section increased by 18.7 Km/h and the congestion length was shortened by 1.4Km. In other perspective, some can say that Weekday bus-only lane policy was settle after the transition period but we only had a monitoring period of 6months, so it's early to say that the policy was fully settled.

## Ⅳ. Conclusion

This study was designed to understand the impacts of the introduction of Weekday Bus-only Lane on Gyeongbu expressway by monitoring changes in traffic. Findings were as follows: First, commuters were more likely to take buses as the public transportation system in Gyeonggi province was improved and the

bus-only lane allowed buses to arrive on time. Second, traffic congestion on other lanes worsened to slow down the speed of passenger cars. Even though traffic nearby Seoul tollgate was reduced, traffic jam in the metropolitan area (between Seoul tollgate and Anseong) got exacerbated. However, the situation is improving after the temporary use of a shoulder lane has been adopted as a short-tem measure to address the congestion in the area.

As such, Weekday Bus-only Lane provides the benefit of better public transportation services, while worsening traffic jam on other lanes. However, the volume of passenger cars is clearly on the decline, suggesting that the congestion will be relieved if the Weekday Bus-only Lane policy is firmly established and more commuters switch to buses. Until now, the severity of traffic jam does not outweigh benefits offered by the bus-only lane, as shown before.

The monitoring in this study initiated when the pilot operation of the bus-only lane started and lasted for 6 months. It is believed that continuous monitoring is required to recognize traffic trend and take proper management measures, because traffic situation changes with newly-opened roads near Gyeongbu expressway and traffic management policies.

## REFERENCE


1. The ministry of land, transport, and maritime, The plan of a weekday bus-only lane policy, 2008
2. The Road Traffic Act, 2008
3. http://www.seoul.go.kr
4. Freeway Traffic Management Systems, Korea Expressway Corporation
5. Toll Collection Systems, Korea Expressway Corporation

# Evaluation of Effect of LCS (Lane Control System) on Expressway

**Lee, Seungjun*, Choi, Yoonhyuk**, Kim, Taeyoung*****

*Transportation Research Division*

*Expressway & Transportation Research Institute*

## Ⅰ. INTRODUCTION

The bus lane in the section from Osan to Hannam on Kyungbu Expressway was implemented in July 1st, 2008 and enforcement to violated vehicles was begun on October. Due to this policy implementation, the number of lane for passenger cars on Kyungbu Expressway was decreased from four to three. On the other hand, total traffic demand was decreased just a little so that lots of congested sections were occurred in the section where bus lane was implemented. In special, in south bound on Kyungbu Expressway, the congestion occurred from Suwon IC reaches to Seoul Toll-gate and the range of congested section is about 12km. To solve this problem, Korea Expressway Corporation performed the study to analyze the cause of congestion and evaluate the effect of improvement alternatives. At the same time, estimation for traffic operating condition after enforcement was performed.

## Ⅱ. Outline of Bus Lane Policy on Kyungbu Expressway

Kyungbu Expressway is symbolic road which has led the economic growth of South Korea.

The Bus Lane policy on Kyungbu Expressway has two aims. One is to decrease severe traffic congestion and the other is to encourage public transportation.

<Figure 1> Location of Kyungbu Expressway in Korea


With those aims, Bus Lane policy was implemented. The process of Bus Lane policy on Kyungbu Expressway is as follows;

- To make a plan for the Bus Lane policy on Kyungbu Expressway (2008. 2)
- To examine the feasibility of Bus Lane policy (2008. 3-6)
- To start Bus Lane policy without enforcement (2008. 7-9)
- To operate Bus Lane policy with enforcement (2008. 10)

1. Comparison of travel demand between before and after Bus Lane policy implementation

Travel demand between before and after Bus Lane policy implementation was compared to know the change of traffic conditions. In the analysis, the used data for before Bus Lane policy implementation was those of early six months and the used data for after Bus Lane policy implementation was those of later six months, because Bus Lane policy was begun in July 2008. Table 1 shows passenger car and total travel demand on Kyungbu Expressway from January to Jun in 2007 and 2008 each. As shown Table 1, the travel demand variation between 2007 and 2008 is not large. On the other hand, Table 2 shows a large amount of travel demand decrease after Bus Lane policy implementation in 2008, compared with travel demand in 2007.

Particularly, as shown in Table 2, passenger car travel demand was decreased very largely. In addition, during the period from September to December, when enforcement was started, travel demand was decrease more than 10% compared with that of previous year.

<Table 1> Variation of travel demand before Bus Lane policy implementation

(Unit: vehicle/day, %)

| | | January | February | March | April | May | Jun |
|---|---|---|---|---|---|---|---|
| Passenger Car | 2007 | 173,192 | 178,668 | 181,124 | 186,718 | 187,530 | 184,737 |
| | 2008 | 171,789 | 179,865 | 184,391 | 186,439 | 187,425 | 181,037 |
| | Rate | -0.81 | 0.67 | 1.80 | -0.15 | -0.06 | -2.00 |
| Total | 2007 | 200,465 | 205,856 | 210,593 | 217,976 | 218,089 | 214,295 |
| | 2008 | 200,310 | 208,281 | 214,688 | 217,434 | 218,432 | 210,608 |
| | Rate | -0.08 | 1.18 | 1.94 | -0.25 | 0.16 | -1.72 |

<Table 2> Variation of travel demand after Bus Lane policy implementation

(Unit: vehicle/day, %)

| | | July | August | September | October | November | December |
|---|---|---|---|---|---|---|---|
| Passenger Car | 2007 | 184,923 | 188,544 | 188,318 | 189,976 | 188,239 | 179,654 |
| | 2008 | 168,418 | 175,015 | 173,396 | 164,221 | 161,902 | 155,229 |
| | Rate | -8.93 | -7.18 | -7.92 | -13.56 | -13.99 | -13.60 |
| Total | 2007 | 212,699 | 215,738 | 218,058 | 221,016 | 218,907 | 208,615 |
| | 2008 | 196,517 | 203,169 | 203,131 | 194,810 | 191,633 | 183,674 |
| | Rate | -7.61 | -5.83 | -6.85 | -11.86 | -12.46 | -11.96 |

2. Comparison of Traffic Condition

1) Average travel speed

Southbound average travel speeds of before and after Bus Lane policy implementation are shown in Table 3. In Table 3, average travel speeds in each section before Bus Lane policy implementation are 94km/h in the section from Seoul Toll to Suwon IC and 84km/h in the section from Suwon IC to Giheung IC. In the period without enforcement, from July to September in 2008, average travel speeds on a Bus Lane in each section are 98km/h and 95km/h. And average travel speeds on General Lanes in each section are 80km/h and 68km/h.

Whereas, In the period with enforcement, from October to December in 2008, average travel speeds on a Bus Lane in each section are 105km/h and 96km/h. And average travel speeds on General Lanes in each section are 71km/h and 63km/h. With these results, it was found that average travel speed on a Bus Lane was increased, but average travel speed on General Lanes was decreased because of Bus Lane policy implementation. Furthermore, through the effect of enforcement, average travel speed on a Bus Lane was much more increased, but average travel speed on General Lanes was much more decreased.

<Table 3> Comparison of travel speed between before and after Bus Lane policy implementation

(Unit: km/h)

<table>
<tr><td colspan="3" rowspan="2"></td><td colspan="2">Seoul Toll ~ Suwon IC</td><td colspan="2">Suwon IC ~ Giheung IC</td></tr>
<tr><td>Bus Lane</td><td>General Lane</td><td>Bus Lane</td><td>General Lane</td></tr>
<tr><td colspan="2">Before</td><td>January ~ Jun</td><td colspan="2">94</td><td colspan="2">84</td></tr>
<tr><td rowspan="2">After</td><td>No enforcement</td><td>July ~ September</td><td>98</td><td>80</td><td>95</td><td>68</td></tr>
<tr><td>Enforcement</td><td>October ~ December</td><td>105</td><td>71</td><td>96</td><td>63</td></tr>
</table>

2) Traffic congestion

As shown in Table 4, the range of average congestion section on Kyungbu Expressway (southbound) before Bus Lane policy implementation was 2.7km, from Singal JC to Suwon IC. On the other hand, the range of average congestion section in the period without enforcement after Bus Lane policy implementation was 7.5km, from Jukjeon to Suwon IC. And also, the range of average congestion section in the period with enforcement after Bus Lane policy implementation was 12km, from Seoul Toll to Suwon IC.

From these results, it was known that the range of average congestion section after Bus Lane policy implementation was increase rapidly compared with that of before Bus Lane policy implementation. Moreover, it is also known that the range of average congestion section in the period with enforcement was more increased than that of in the period without enforcement.

<Table 4> Comparison of traffic congestion section between before and after Bus Lane policy implementation

<table>
<tr><td colspan="2"></td><td>Congestion section</td><td>Maximum congestion</td></tr>
<tr><td>Before</td><td>January ~ Jun</td><td>Singal JC ~ Suwon IC</td><td>2.7km</td></tr>
<tr><td rowspan="2">After</td><td>July ~ September<br>(No enforcement)</td><td>Jukjeon ~ Suwon IC</td><td>7.5km</td></tr>
<tr><td>October ~ December<br>(Enforcement)</td><td>Seoul Toll ~ Suwon IC</td><td>12km</td></tr>
</table>

3. Analysis of traffic congestion problem

Traffic congestion on Kyungbu Expressway (southbound) was caused due to merging flow on Suwon IC, so it could be said that Suwon IC merging section was operated as a bottle-neck

section on Kyungbu Expressway (southbound). From the Bus Lane policy implementation, the effect of traffic congestion occurred from Suwon IC was up to Seoul Toll.

It can be said that the level of traffic congestion at present like this is the level of equilibrium condition. That is, through the Bus Lane policy implementation, much more severe traffic congestion was occurred than before and thereafter considerable traffic demand was decreased and present traffic equilibrium condition was reached.

## Ⅲ. Analysis of Effect of LCS (Lane Control System)

After Bus Lane policy implementation without enforcement, there was slightly severe traffic congestion and it was expected that much more severe traffic congestion would occur in the period with enforcement. So, the exertion to find out the cause of traffic problem was done by the field survey and traffic data collection and analysis. After this, another exertion to search for alternatives for improvement was followed.

As a result, it was found that the merging area of Suwon IC caused the congestion because a lot of vehicles entered into main line from on-ramp of Suwon IC. And also, it was found that enlargement of main line section from Suwon IC merging area to Giheung IC diverging area because a lot of vehicles exited into off-ramp of Giheung IC from main line could be effective to reduce the traffic congestion and robust improvement alternative. Finally, after examinations

for various alternatives, it was concluded that enlargement of main line section from on-ramp of Suwon IC to off-ramp of Giheung IC was robust alternative.

Traffic analysis program used for the research was ExTrAM (Expressway Traffic Analysis Model) which was developed by the researchers of KEC (Korea Expressway Corporation).

1. Assumption and Precondition

□ Alternative

- Operation of LCS through enlargement of main line from on-ramp of Suwon IC to off-ramp of Giheung IC

□ Violation rate of other vehicles on Bus Lane

- 20% of Bus Lane volume

□ Classification of analysis cases

- Case 1: Bus Lane policy implementation without enforcement
- Case 2: Bus Lane policy implementation with enforcement - in this case, violation traffic, 20% of Bus Lane volume moves to General Lane from Bus Lane

2. Analysis Results

□ **Present traffic condition**

Figure 2 shows travel speed in each time period on Kyungbu Expressway (southbound) from Seoul Toll to Giheung IC for weekdays during the period of Bus Lane policy implementation without enforcement. As shown in Figure 2, traffic congestion was occurred from Suwon IC and reached to Jukjeon. Maximum average traffic congestion is about 7.5km.

□ **Analysis: Case 1**

As shown in Table 5 and Figure 3, the analysis results of case 1 is as follows;

○ Do Nothing

- Maximum range of traffic congestion: 8.5km
- Total delay cost: 179 million Won/Day (45,014 million Won/Year)

○ Do Alternative

- No traffic congestion

□ **Analysis: Case 2**

As shown in Table 5 and Figure 4, the analysis results of case 2 is as follows;

○ Do Nothing

- Maximum range of traffic congestion: 11.8km
- Total delay cost: 305 million Won/Day (76,763 million Won/Year)

○ Do Alternative

- No traffic congestion

<Figure 2> Present Traffic Condition (Seoul Toll → Giheung IC)

구간별 지정체도(2008. 9. 1(월))-속도

평균 : 속도

| 구간 | 이정(km) | 06:00 | 07:00 | 08:00 | 09:00 | 10:00 | 11:00 | 12:00 | 13:00 | 14:00 | 15:00 | 16:00 | 17:00 | 18:00 | 19:00 | 20:00 | 21:00 | 22:00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 수원IC->기흥IC | 390.75 | 95 | 84 | -999 | 96 | -999 | -999 | 42 | 90 | 94 | 104 | 104 | 110 | 107 | 93 | 77 | 80 | 77 |
| | 391.84 | 92 | 69 | -999 | 73 | -999 | -999 | 73 | 76 | 74 | 72 | 78 | 82 | 83 | 79 | 79 | 81 | 82 |
| | 393.31 | 90 | 64 | -999 | -999 | -999 | -999 | 57 | 56 | 63 | 62 | 66 | 81 | 85 | 79 | 77 | 79 | 78 |
| | 394.46 | 95 | 46 | -999 | 38 | -999 | -999 | 39 | 40 | 38 | 39 | 63 | 86 | 86 | 83 | 84 | 87 | 86 |
| 신갈JC->수원IC | 395.85 | 103 | 51 | -999 | 20 | -999 | -999 | -999 | 26 | 32 | 37 | 83 | 92 | 93 | 82 | 78 | 86 | 88 |
| | 396.70 | 89 | 64 | -999 | 25 | -999 | -999 | 25 | 23 | 27 | 59 | 75 | 81 | 78 | 70 | 70 | 75 | 75 |
| 서울TG->신갈JC | 397.79 | 103 | 87 | -999 | 23 | -999 | -999 | 22 | 23 | 38 | 90 | 94 | 96 | 94 | 82 | 82 | 87 | 88 |
| | 401.43 | 73 | 74 | -999 | 26 | -999 | -999 | 60 | 63 | 88 | 67 | 89 | 79 | 100 | 72 | 81 | 82 | 91 |
| | 402.52 | -999 | 98 | -999 | -999 | -999 | -999 | 36 | -999 | 85 | 83 | 97 | -999 | -999 | -999 | 99 | 97 | 98 |
| | 403.79 | 100 | 73 | -999 | 51 | -999 | -999 | 40 | 93 | 87 | 72 | 80 | 82 | 82 | 74 | 77 | 81 | 82 |
| | 405.13 | -999 | 74 | -999 | 62 | -999 | -999 | 78 | -999 | -999 | -999 | -999 | 84 | 83 | -999 | -999 | -999 | 79 |

구간별 지정체도(2008. 9. 2(화))-속도

평균 : 속도

| 구간 | 이정(km) | 06:00 | 07:00 | 08:00 | 09:00 | 10:00 | 11:00 | 12:00 | 13:00 | 14:00 | 15:00 | 16:00 | 17:00 | 18:00 | 19:00 | 20:00 | 21:00 | 22:00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 수원IC->기흥IC | 390.75 | 90 | 110 | 111 | 107 | 108 | -999 | 109 | 113 | 108 | 111 | 99 | 111 | 120 | 134 | 99 | 91 | 90 |
| | 391.84 | 95 | 85 | 77 | 81 | 80 | 81 | 80 | 82 | 77 | 79 | 70 | 85 | 84 | 90 | 89 | 89 | 88 |
| | 393.31 | 96 | 83 | 68 | 64 | 66 | 65 | 64 | 66 | 65 | 62 | 69 | 84 | 87 | 91 | 88 | 89 | 88 |
| | 394.46 | 99 | 86 | 59 | 45 | 41 | 35 | 37 | 39 | 41 | 40 | 72 | 87 | 89 | 93 | 92 | 93 | 91 |
| 신갈JC->수원IC | 395.85 | 107 | 94 | 67 | 41 | 39 | 24 | 26 | 33 | 40 | 39 | 83 | 98 | 101 | 98 | 92 | 97 | 97 |
| | 396.70 | 92 | 82 | 60 | 44 | 32 | 26 | 28 | 28 | 27 | 29 | 69 | 82 | 82 | 82 | 81 | 83 | 84 |
| 서울TG->신갈JC | 397.79 | 106 | 99 | 94 | 90 | 70 | 20 | 24 | 31 | 30 | 42 | 88 | 106 | 108 | 104 | 98 | 100 | 100 |
| | 401.43 | 62 | 76 | 72 | 78 | 78 | 69 | 26 | 27 | 57 | 93 | 70 | 83 | 83 | 83 | 76 | 81 | 83 |
| | 402.52 | -999 | -999 | 90 | -999 | 100 | -999 | 38 | 37 | 61 | 99 | 78 | 93 | 99 | 94 | 91 | 95 | 95 |
| | 403.79 | 104 | 88 | 77 | 83 | 79 | 84 | 35 | 30 | 74 | 94 | 75 | 90 | 93 | 89 | 88 | 92 | 93 |
| | 405.13 | 92 | -999 | -999 | -999 | -999 | 78 | -999 | 45 | 89 | 90 | 73 | 87 | 91 | 89 | 86 | 88 | 88 |

구간별 지정체도(2008. 9. 3(수))-속도

평균 : 속도

| 구간 | 이정(km) | 06:00 | 07:00 | 08:00 | 09:00 | 10:00 | 11:00 | 12:00 | 13:00 | 14:00 | 15:00 | 16:00 | 17:00 | 18:00 | 19:00 | 20:00 | 21:00 | 22:00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 수원IC->기흥IC | 390.75 | 104 | 120 | 98 | 92 | 98 | 105 | 115 | 117 | 110 | 112 | 115 | 125 | 129 | 132 | 95 | 88 | 88 |
| | 391.84 | 97 | 87 | 80 | 80 | 78 | 77 | 76 | 78 | 77 | 77 | 80 | 85 | 88 | 90 | 89 | 88 | 88 |
| | 393.31 | 96 | 87 | 70 | 68 | 61 | 61 | 61 | 64 | 60 | 66 | 77 | 86 | 89 | 91 | 89 | 88 | 88 |
| | 394.46 | 100 | 90 | 63 | 57 | 41 | 39 | 39 | 36 | 42 | 49 | 70 | 88 | 92 | 93 | 92 | 91 | 91 |
| 신갈JC->수원IC | 395.85 | 109 | 100 | 69 | 72 | 38 | 33 | 29 | 32 | 41 | 54 | 87 | 100 | 102 | 98 | 93 | 93 | 96 |
| | 396.70 | 98 | 85 | 67 | 32 | 32 | 29 | 25 | 27 | 37 | 29 | 75 | 83 | 85 | 83 | 81 | 82 | 84 |
| 서울TG->신갈JC | 397.79 | 114 | 107 | 99 | 60 | 72 | 31 | 27 | 31 | 76 | 61 | 104 | 105 | 107 | 105 | 100 | 98 | 101 |
| | 401.43 | 80 | 93 | 80 | 85 | 73 | 68 | 37 | 52 | 53 | 87 | 85 | 82 | 87 | 82 | 76 | 78 | 80 |
| | 402.52 | 108 | 97 | 90 | 79 | 81 | 78 | 55 | 75 | 63 | -999 | 95 | 93 | -999 | 93 | -999 | 90 | 93 |
| | 403.79 | 106 | 91 | 81 | 86 | 74 | 65 | 60 | 90 | 66 | 92 | 92 | 91 | 92 | 90 | 91 | 86 | 93 |
| | 405.13 | 97 | 88 | 80 | 84 | 71 | 52 | 63 | 87 | 74 | 87 | 90 | 91 | 91 | 90 | 89 | 85 | 89 |

구간별 지정체도(2008. 9. 4(목))-속도

평균 : 속도

| 구간 | 이정(km) | 06:00 | 07:00 | 08:00 | 09:00 | 10:00 | 11:00 | 12:00 | 13:00 | 14:00 | 15:00 | 16:00 | 17:00 | 18:00 | 19:00 | 20:00 | 21:00 | 22:00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 수원IC->기흥IC | 390.75 | 102 | 121 | 107 | 105 | 39 | 107 | 113 | 119 | 110 | 119 | -999 | 117 | 124 | 129 | 92 | 89 | 89 |
| | 391.84 | 97 | 88 | 78 | 80 | 27 | 75 | 76 | 77 | 76 | 81 | -999 | 84 | 87 | 87 | 87 | 89 | 88 |
| | 393.31 | 97 | 87 | 63 | 65 | 23 | 62 | 62 | 63 | 62 | 75 | -999 | 87 | 89 | 88 | 87 | 89 | 88 |
| | 394.46 | 99 | 89 | 54 | 43 | 15 | 38 | 36 | 36 | 39 | 73 | -999 | 88 | 89 | 91 | 91 | 92 | 91 |
| 신갈JC->수원IC | 395.85 | 81 | 95 | 69 | 37 | 13 | 31 | 27 | 30 | 41 | 88 | -999 | 97 | 98 | 97 | 87 | 95 | 96 |
| | 396.70 | 94 | 85 | 52 | 33 | 10 | 29 | 27 | 27 | 29 | 76 | -999 | 79 | 82 | 81 | 78 | 82 | 84 |
| 서울TG->신갈JC | 397.79 | 116 | 109 | 100 | 82 | 14 | 32 | 28 | 29 | 45 | 103 | -999 | 101 | 104 | 103 | 97 | 102 | 100 |
| | 401.43 | 70 | 28 | 45 | 76 | 23 | 71 | 54 | 66 | 84 | 81 | -999 | 79 | 83 | 83 | 76 | 85 | 82 |
| | 402.52 | 109 | 100 | 96 | 97 | -999 | 78 | 77 | 90 | -999 | -999 | -999 | 86 | 84 | 84 | 89 | 97 | -999 |
| | 403.79 | 107 | 94 | 91 | 92 | 29 | 72 | 75 | 89 | 91 | 92 | -999 | 88 | 90 | 88 | 87 | 96 | 93 |
| | 405.13 | 98 | 91 | 86 | 88 | 31 | 62 | 62 | 86 | 88 | 90 | -999 | 89 | 89 | 89 | 85 | 89 | 89 |

구간별 지정체도(2008. 9. 5(금))-속도

평균 : 속도

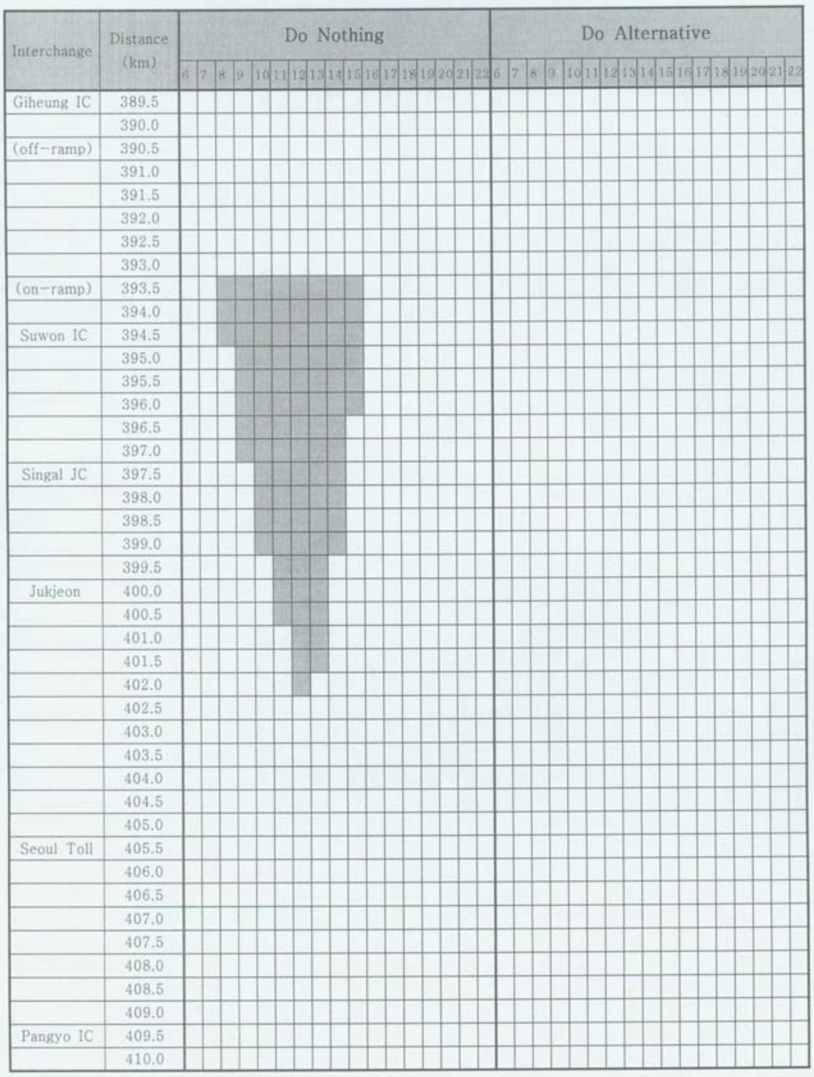
| 구간 | 이정(km) | 06:00 | 07:00 | 08:00 | 09:00 | 10:00 | 11:00 | 12:00 | 13:00 | 14:00 | 15:00 | 16:00 | 17:00 | 18:00 | 19:00 | 20:00 | 21:00 | 22:00 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 수원IC->기흥IC | 390.75 | 99 | 124 | 109 | 115 | 112 | 110 | 109 | 110 | 109 | 111 | 114 | 115 | 117 | 117 | 86 | 86 | 91 |
| | 391.84 | 97 | 87 | 79 | 80 | 80 | 74 | 76 | 74 | 77 | 76 | 79 | 83 | 84 | 85 | 83 | 85 | 90 |
| | 393.31 | 97 | 88 | 72 | 72 | 74 | 63 | 64 | 63 | 64 | 60 | 70 | 81 | 83 | 84 | 81 | 84 | 35 |
| | 394.46 | 100 | 90 | 60 | 63 | 68 | 40 | 38 | 37 | 38 | 39 | 61 | 82 | 85 | 87 | 87 | 87 | 44 |
| 신갈JC->수원IC | 395.85 | 107 | 98 | 76 | 78 | 91 | 42 | 32 | 29 | 32 | 40 | 77 | 97 | 97 | 93 | 83 | 88 | 61 |
| | 396.70 | 95 | 84 | 75 | 78 | 76 | 42 | 28 | 28 | 27 | 33 | 76 | 83 | 83 | 80 | 75 | 79 | 75 |
| 서울TG->신갈JC | 397.79 | 113 | 107 | 98 | 100 | 99 | 75 | 30 | 39 | 40 | 84 | 101 | 105 | 104 | 102 | 95 | 96 | 98 |
| | 401.43 | 66 | 82 | 60 | 84 | 83 | 76 | 69 | 68 | 74 | 78 | 89 | 89 | 91 | 91 | 83 | 82 | 76 |
| | 402.52 | 108 | 97 | 84 | 89 | 86 | 79 | 76 | 73 | 82 | 87 | 98 | 99 | 99 | 98 | 90 | 93 | 93 |
| | 403.79 | 108 | 93 | 68 | 84 | 77 | 65 | 56 | 63 | 77 | 84 | 94 | 94 | 94 | 93 | 89 | 90 | 91 |
| | 405.13 | 98 | 90 | 83 | 87 | 84 | 62 | 42 | 63 | 85 | 86 | 89 | 90 | 90 | 89 | 82 | 84 | 85 |

<Table 5> Analysis Results of LCS Effect

| Classification | MOE | Alternative | | **Effects** |
|---|---|---|---|---|
| | | Do Nothing | Do Alternative | |
| **Case 1** | Maximum Traffic Congestion (km) | 8.5 | - | 8.5 |
| | Total Delay Time (hour) | 12,481 | - | 12,481 |
| | Total Delay Cost (million ₩) | 179 (Day)<br>45,014 (Year) | -<br>- | 179 (Day)<br>**45,014 (Year)** |
| **Case 2** | Maximum Traffic Congestion (km) | 11.8 | - | 11.8 |
| | Total Delay Time (hour) | 21,284 | - | 21,284 |
| | Total Delay Cost (million ₩) | 305 (Day)<br>76,763 (Year) | -<br>- | 305 (Day)<br>**76,763 (Year)** |

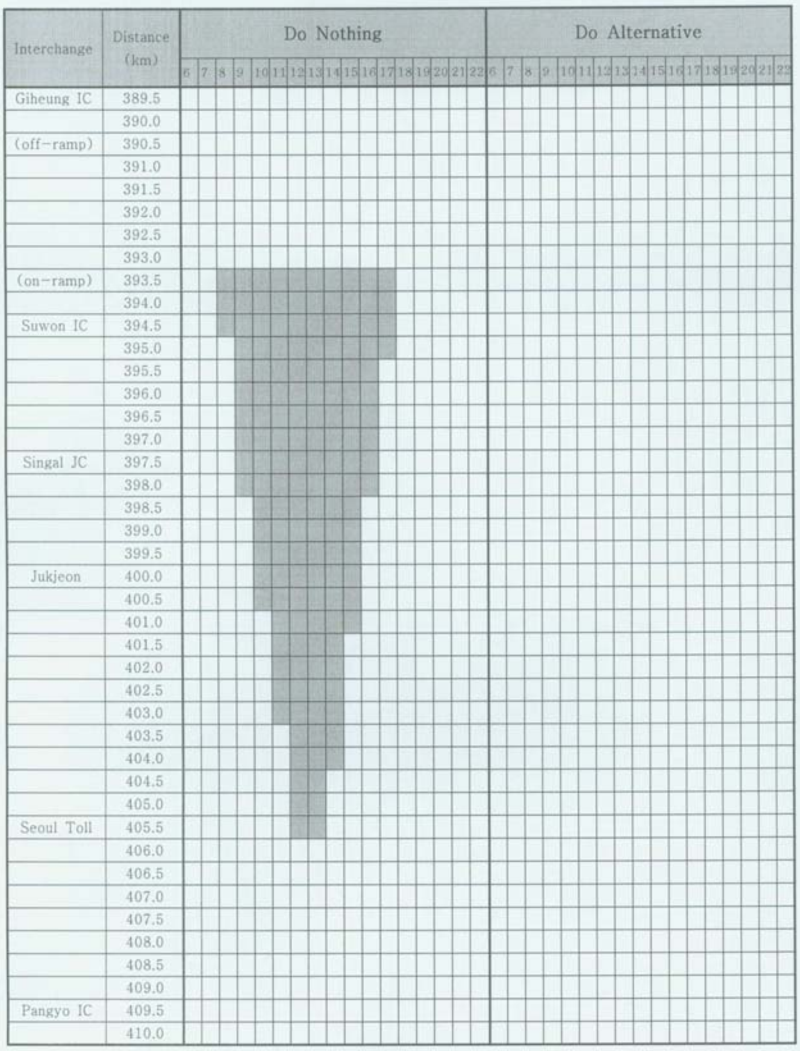
<Figure 3> Congestion Analysis Result in a Time-Space Diagram: Case 1

<Figure 3> Congestion Analysis Result in a Time-Space Diagram : Case 1



<Figure 4> Congestion Analysis Result in a Time-Space Diagram: Case 2

<Figure 4> Congestion Analysis Result in a Time-Space Diagram : Case 2



## Ⅳ. The Effect of LCS Implementation

KEC (Korea Expressway Corporation) implemented LCS on Kyungbu Expressway (southbound) from on-ramp of Suwon IC to off-ramp of Giheung IC in December 2008 based on the previous analysis results. As a result, the congestion between Suwon IC and Seoul Toll-gate was disappeared completely. After this, KEC are preparing another LCS implementation for other congestion sections on Kyungbu Expressway (northbound). Other congestion sections being prepared LCS implementation are as follows; from Giheung IC to Suwon IC, from Singal JC to Seoul Toll, from Seoul Toll to Pangyo IC.

## Ⅴ. Conclusion

Bus Lane policy was implemented for traffic congestion reduction and public transportation, but there were some problems because it was implemented without pre-examination on traffic condition. So, after Bus Lane policy implementation, the exertions to find out the cause of congestion problem and the effective alternative were performed. Fortunately, implementation of LCS on congested section had a great effect to reduce traffic congestion. However, a lot of money had to be paid for it. Therefore, it is needed to note that examination on any transportation policy before implementation has to be performed always.

□ REFERENCE

1. California Department of Transportation Division of Research & Innovation, Transportation System Impacts Cross- cutting Evaluation Report, 2004.

2. CAMPO, “Congestion management system (CMS) state of the system report”, 2004.

3. Chao Chen, “Cause and Cures of Highway Congestion”, 2001.

4. FHWA, “Freeway Management and Operations Handbook”, 2003.

5. Pravin Varaiya, “How to measure Transportation system performance”, 1997.

6. Pravin Varaiya, “Freeway Performance Measurement System (PeMS), Version 3 Phase Ⅱ,” 2004.

7. Quantifying congestion, NCHRP Report 398 Vol 1, 1997.

8. Texas Transportation Institute, “Monitoring Urban Freeways in 2003: Current Conditions and Trends from Archived Operations Data”, 2004.

9. Texas Transportation Institute, “2005 Urban Mobility Report,” 2005.

10. Transportation Research Board, “FREEWAY PERFORMANCE MEASUREMENT SYSTEM: MINING LOOP DETECTOR DATA”, 2001.

# Study on the Design Method Development and Impact Analysis of Crash Cushion Using Single Degree of Freedom

**Lee Jae Seong[1], Joo, Jae Woong[2, *], Kum, Ki Jung[3], Jang, Dae Young[4] , Lee, Yoon Ki[5]**

[1]*Deputy General Manager, Korea Expressway Corporation, Expressway & Transportation Research Institute*
[1]*Associate Research Engineer, Korea Expressway Corporation, Expressway & Transportation Research Institute*
[3]*Professor, Department of Transportation Engineering, Myongji University*
[4]*Researcher, Korea Expressway Corporation, Expressway & Transportation Research Institute*
[5]*Professor, Department of Automobile Engineering, Chosun College of Science*

## Abstract

Crash cushion is a safety facility that can make a direction restore to the original lane or make it stop by absorbing the impact energy of vehicle. But development of crash cushion is defective because there's no rational and reality way of design. And also without an alternative plan, it rely on crash test hereby it suffers a great economic loss and wastes time.

This study proves the suitability of single degree of freedom considered passenger safety and performs design development of effective crash cushion to analyze behavior of collision of crash cushion. To verify the validness of the crash cushion design method, the researchers execute the crash test. A performance test brings satisfied result and judging from this, the design method of single degree of freedom is proved one of the best ways to design crash cushion

Key words : Crash Cushion, Design Method, Impact Analysis, Crash Test, Occupant Risk Index

## 1. Introduction

Crash cushion is a safety facility that can make a direction restore to the original lane or make it stop by absorbing the impact energy of vehicle. In Korea these days, many kinds of crash cushions have been positioned at several spots where vehicle collision is most likely to happen, like a bridge post, abutments of a bridge, a junction of a linking road exit, a tollgate, a tunnel and an underpass entrance.

According to the rules in KOREA, a crash cushion should pass standard vehicle crash test. For development of crash cushion, efficiency is completed the crash test. However, there is no reasonable method that considers passenger's safety and only depends on crash test without an alternative plan. Recently, numerical interpretation programs are used variously but modeling the structure of both the crash cushion and the car is complicated & difficult, resulting to high computational cost. Consequently the researchers are asked to develop a systematic design method.

For the design of facilities of the crash cushion considering passenger safety, this paper analyzes the behavior of collision of the crash cushion facility based on data of 34 crash tests. Hereby, the researchers conducted a basic survey that helps the development of the crash cushion. The suitability of single degree of freedom is verified through the comparison of passenger safety indexes. Crash test data(x-axis acceleration, y-axis acceleration, yaw angular velocity) are applicable to the passenger safety index, for the calculation of theoretical head impact velocity(THIV) and post-impact head deceleration(PHD). And other passenger safety index is calculated only x-axis acceleration.

* Corresponding author
Tel : +82-031-371-3318 , Fax : +82-031-371-3249
E-mail : jjw123@ex.co.kr

It is better to use the data of a vehicle crash test rather than using data of crash simulation on crash cushion in developing the safety crash cushion. On this study, crash test data of verified crash cushion CC1 level have been applied to single degree of freedom's design, and then have planned crash cushion CC2 level and CC3. Each level crash cushion used in this study is identical in character of critical structure and only has some differences in length considering validity transformational distance.

All of the crash cushion's main part is consist of steel in general. Physical property of the steel controls efficiently vehicle's impact load and is suitable improving for the facility’s crash absorption performance. For developing of the crash cushion was ahead of steel structure analysis. On this study has developed crash cushion's design method stand on steel structure analysis.

This study proves the suitability of single degree of freedom considered passenger safety and performs design development of effective crash cushion to analyze behavior of collision of crash cushion.

## 2. Theoretical consideration on single degree of freedom

When a vehicle collides with the crash cushion, preserved part of the vehicle is assumed as a rigid body. After vehicle collision, impact energy was absorbed by the damaged part of crash cushion. If a constant spring which is resisting force about transformation of crash cushion is considered, impact of vehicle and crash cushion may be represented by a single degree of freedom as shown in figure 1 using a constant spring $k$ , resisting force, which is generated by impact of crash cushion and mass of the vehicle, $M$ .

The time of collision, the vehicle and crash cushion's character is nonlinear and the character is known easily as showing a graph that represents relationship between resisting force and displacement of the vehicle when collided.

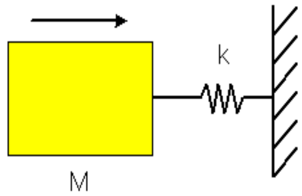



Figure 1. Crash cushion's model picture of single degree of freedom

The resisting force is found by multiplying mass of a vehicle and acceleration of a vehicle measured preserved part, a time record of displacement while crash is found by integrating measured acceleration twice. Suppose that mass of impact vehicle, $M$ , acceleration measured the part of preserved vehicle, $a$ , outside force like friction during collision, 0, then movement equation is organized.

$$f = Ma = 0 \tag{1}$$

When the outside force is 0, velocity of vehicle and displacement is equation (2), (3).

$$v = v_0 + \int_0^t adt \tag{2}$$

$$x = v_0 t + \int_0^{t'} \int_0^t adtdt' \tag{3}$$

This displacement $x$ the represents displacement of a vehicle body and structure, velocity $v_0$ represents initial velocity, $t$ , $t'$ is the time on acceleration and velocity. Plus under ignorance of vehicle's displacement circumstances, displacement $x$ sees displacement of structure. Upper process can show the graph of displacement and acceleration( $x$ vs. $a$ ), the unit mass resisting force, applying to the result of simulation or crash test.

According to R. I. Emori study, complex oscillation the relationship between resisting force and displacement generating under the impact process of the vehicle and a rigid wall seems severe curves but becomes clear that it can be closely aligned at a fixed incline. Namely Emori analyzed the data of crash test, and then reached the result that a

unit mass of constant spring ( $k/M$ ) has the constant value regardless of the impact velocity. That means that a rate of measured acceleration and displacement( $a/x$ ), is constant. The figure 2 sees the acceleration and displacement. (Emori, 1968)

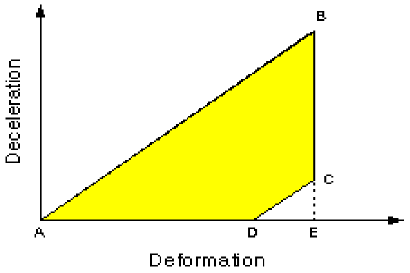



Figure 2. An ideal spring of single degree of freedom

Colored area in figure 2 shows movement energy of the unit mass of vehicle that is absorbed by nonlinear spring. When displacement of spring showed impact character of vehicle is $x$ , resisting force $f$ sees $kx$ . Moreover, $f = kx = Ma$ , then can be described by Newton's movement law. Since $k/M = a/x$ is affected, if $a/x$ is constant, equally $k/M$ is also constant. Because $k/M$ is same as natural frequency of system, $\omega^2$ , the movement equation of vehicle using single degree of freedom model can be written like this. (Go Man-ki, 2003)

$$\frac{a}{x} = \frac{-\ddot{x}}{x} = -\frac{k}{M} = -\omega^2 \tag{4}$$

$$M\ddot{x} = kx = 0 \tag{5}$$

At this point, let us supposes initial velocity , initial displacement 0.

$$\text{Displacement} : \; x = \frac{v_0}{\omega}\sin\omega t \quad (0 < \omega t < \frac{\pi}{2}),$$

$$x = 0 \quad (\frac{\pi}{2} \le \omega t) \tag{6}$$

$$\text{Velocity} : \; \dot{x} = v_0 \cos\omega t \tag{7}$$

$$\text{Acceleration} : \; \ddot{x} = -v_0\omega \sin\omega t \tag{8}$$

Therefore, the maximum displacement and acceleration is represented a function of initial velocity .

$$|\ddot{x}_{\max}| = \omega v_0 \tag{9}$$

$$x_{\max} = \frac{v_0}{\omega} \tag{10}$$

If the character of vehicle-rigid wall impact can confirmed from the vehicle-crash cushion impact, design of reasonable crash cushion that considers passenger safety is quite possible. If rate of acceleration measured the vehicle-crash cushion impact and displacement coming from the acceleration( $a/x$ ) is confirmed in regardless of velocity, crash cushion's maximum displacement and the maximum acceleration can represent function of impact velocity. Vehicle transforming displacement that is compared with crash cushion's transforming displacement is much small, so it is neglected, and the maximum acceleration means X-axis maximum acceleration which is direction of impact.

## 3. Test condition and evaluation criteria of crash cushion

In the world, test condition and evaluation criteria of crash cushion are applied by considering each country's road condition and evaluation criteria is divided in USA NCHRP Report 350 and European Standard(CEN). Korean standard is the reorganized European standard and the condition of crash test sees in figure 3 and table 1.

According to KOREA guide, the standard of evaluation is divided in passenger protection capacity, behavior of crash cushion & behavior of vehicle after collision. There are two evaluation standards passenger collision velocity(THIV) and passenger acceleration(PHD). In case of THIV, test ①, ②, ③ should be below 44km/h , test ④, ⑤ below 33km/h and PHD should be below 20g.

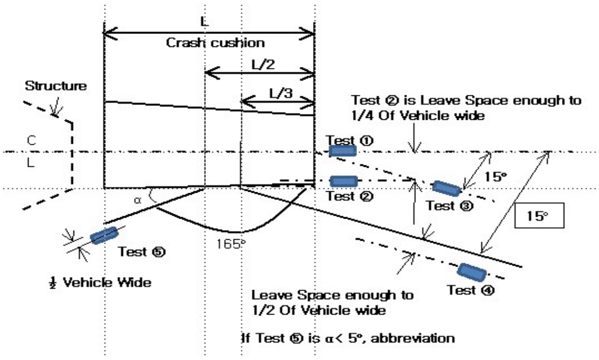



Figure 3. Collision spot and direction of collision vehicle

Table 1. Condition of collision test

| Level | Collision velocity (Km/h) | total vehicle mass (Kg) | collision method |
|---|---|---|---|
| CC1 | 60 | 900<br>1,300 | test ①<br>test ② |
| CC2 | 80 | 900<br>1,300 | test ①, ②<br>test ①, ③, ④, ⑤ |
| CC3 | 100 | 900<br>1,300 | test ①, ②<br>test ①, ③, ④, ⑤ |

## 4. Collision analysis of crash cushion for applying single degree of freedom

This study is analyzed crash test data that are executed 34 times in the Expressway & Transportation research Institute from Sep. 2005 to June. 2007. In here, this paper has excluded broadside collision(No ④, ⑤) data because passenger safety of broadside collision is shown very safe and broadside collision is not a test for passenger safety but for behavior of vehicle-crash cushion and scattering efficiency. Stage of crash cushion design considering passenger safety must be designed with an eye to head on collision rather than broadside collision.

Table 2 compares THIV and PHD that are result of head on collision and 15° broadside collision test based on identical crash cushion.

Test No ④`s THIV average is about Test No ① 40.2% and PHD shows relatively stable result 30.8%. Standard of PHD is provided under the 20g, Test ①, ④ but THIV Test ① should be under the 44km/h, Test ④ under the 33km/h. Therefore absolute comparison is unreasonable. Therefore, a THIV average of test No ① shows 87.8% of a valuation basis(44km/h) but even average of test No ④ shows 47.1% of a valuation basis(33km/h).

Therefore, crash cushion design considering passenger safety should focus on head on collision rather than broad side collision. 34 kinds of crash cushion used in the analysis presented difference in structural mechanism absorbing a collision load of a vehicle, an application level, a collision test method, collision vehicle weight, a size and material.

According to Korean guide, THIV is calculated with x-axis acceleration, y-axis acceleration and yaw angular velocity, PHD is calculated with x-axis acceleration, y-axis. Table 4, 5 compare calculation result by THIV and PHD.

Table 2. Comparison of occupant risk index head on collision & broad side collision

| Test No. | Test Level | 0.9ton, head on collision (Test No ①) | | 1.3ton 15°, side collision (Test No ④) | |
|---|---|---|---|---|---|
| | | THIV (Under 44km/h) | PHD (Under 20g ) | THIV (Under 33km/h ) | PHD (Under 20g) |
| Test 1 | CC1 | 31.0 | 14.1 | 16.2 | 10.1 |
| Test 2 | CC1 | 33.7 | 30.3 | 14.9 | 1.1 |
| Test 3 | CC1 | 43.7 | 18.3 | 14.0 | 1.4 |
| Test 4 | CC1 | 39.9 | 11.0 | 16.8 | 1.8 |
| Test 5 | CC1 | 38.0 | 18.5 | 12.4 | 5.1 |
| Test 6 | CC1 | 40.2 | 12.6 | 15.9 | 9.1 |
| Test 7 | CC2 | 43.8 | 13.0 | 18.5 | 7.7 |
| Average | | 38.61 | 16.83 | 15.53 | 5.19 |

Table 3. Comparison of cc1 crash cushion's occupant risk index

| Test No | condition | result | calculated all of X, Y, Yaw | | | | Calculated of X-axis acceleration | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | THIV | | PHD | | THIV | | PHD | |
| | | | Km/hr | sec | g's | sec | Km/hr | sec | g's | sec |
| No.01-CC1 | ①-0.9 F | Good | 43.7100 | 0.1152 | 18.3284 | 0.1570 | 43.7086 | 0.1152 | 18.2601 | 0.1569 |
| No.02-CC1 | ①-0.9 F | Good | 41.6008 | 0.1169 | 16.0170 | 0.1574 | 41.4627 | 0.1169 | 15.9295 | 0.1574 |
| No.03-CC1 | ①-0.9 F | Good | 39.9431 | 0.1206 | 11.0450 | 0.1564 | 39.9409 | 0.1206 | 11.0449 | 0.1564 |
| No.04-CC1 | ①-0.9 F | Good | 35.4276 | 0.1227 | 18.5250 | 0.1561 | 35.4268 | 0.1227 | 18.5249 | 0.1561 |
| No.05-CC1 | ①-0.9 F | Good | 38.0454 | 0.1116 | 18.4745 | 0.1543 | 38.0292 | 0.1116 | 18.4556 | 0.1543 |
| No.06-CC1 | ①-0.9 F | Good | 40.1771 | 0.1186 | 12.6479 | 0.1358 | 40.1411 | 0.1186 | 12.5734 | 0.1356 |
| No.07-CC1 | ①-0.9 F | NG | 30.8741 | 0.1446 | 43.9931 | 0.1735 | 30.8711 | 0.1446 | 43.9431 | 0.1736 |
| No.08-CC1 | ①-0.9 F | NG | 42.1811 | 0.1147 | 24.2970 | 0.1335 | 42.1787 | 0.1147 | 24.2337 | 0.1335 |
| No.09-CC1 | ①-0.9 F | NG | 52.8988 | 0.0953 | 29.6503 | 0.0996 | 52.8546 | 0.0953 | 29.2760 | 0.0995 |
| No.10-CC1 | ①-0.9 F | NG | 57.2705 | 0.0994 | 18.9891 | 0.0994 | 57.2336 | 0.0994 | 18.8352 | 0.0994 |

Table 4. Comparison of cc2 crash cushion's occupant risk index

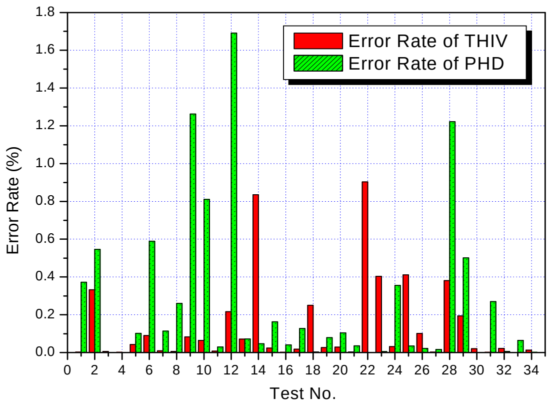
| Test No | condition | result | Calculated all of X, Y, Yaw | | | | Calculated of X-axis acceleration | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | THIV | | PHD | | THIV | | PHD | |
| | | | Km/hr | sec | g's | sec | Km/hr | sec | g's | sec |
| No.11-CC2 | ①-0.9 F | Good | 42.0251 | 0.0973 | 10.6033 | 0.2886 | 42.0218 | 0.0973 | 10.6001 | 0.2886 |
| No.12-CC2 | ①-0.9 F | Good | 42.4902 | 0.0938 | 9.6143 | 0.0981 | 42.3983 | 0.0938 | 9.4517 | 0.0979 |
| No.13-CC2 | ①-0.9 F | Good | 43.8156 | 0.1004 | 13.0085 | 0.1266 | 43.7844 | 0.1004 | 12.9991 | 0.1266 |
| No.14-CC2 | ②-0.9 O | Good | 38.8899 | 0.1016 | 16.2040 | 0.3013 | 38.5649 | 0.1016 | 16.1964 | 0.3013 |
| No.15-CC2 | ②-0.9 O | Good | 43.1883 | 0.1034 | 15.2749 | 0.1807 | 43.1780 | 0.1034 | 15.2500 | 0.1808 |
| No.16-CC2 | ①-1.3 F | Good | 37.4017 | 0.1034 | 16.0357 | 0.1153 | 37.4022 | 0.1034 | 16.0291 | 0.1153 |
| No.17-CC2 | ①-1.3 F | Good | 35.2950 | 0.1138 | 18.6516 | 0.2632 | 35.2886 | 0.1138 | 18.6278 | 0.2633 |
| No.18-CC2 | ①-1.3 F | Good | 33.9583 | 0.1167 | 17.0224 | 0.2990 | 33.8733 | 0.1167 | 17.0218 | 0.2990 |
| No.19-CC2 | ①-1.3 F | Good | 34.5902 | 0.1191 | 15.3571 | 0.2735 | 34.5809 | 0.1191 | 15.3450 | 0.2735 |
| No.20-CC2 | ①-1.3 F | Good | 33.8304 | 0.1192 | 13.4172 | 0.3114 | 33.8207 | 0.1192 | 13.4032 | 0.3096 |
| No.21-CC2 | ①-1.3 F | Good | 40.4505 | 0.1042 | 11.6040 | 0.1159 | 40.4490 | 0.1042 | 11.5998 | 0.1159 |
| No.22-CC2 | ③-1.3 D | Good | 36.1304 | 0.1136 | 15.0163 | 0.1788 | 35.8039 | 0.1136 | 15.0163 | 0.1788 |
| No.23-CC2 | ③-1.3 D | Good | 39.7278 | 0.1139 | 13.0194 | 0.2255 | 39.5674 | 0.1139 | 13.0187 | 0.2255 |
| No.24-CC2 | ①-0.9 F | NG | 46.1142 | 0.0918 | 9.1891 | 0.2937 | 46.1287 | 0.0918 | 9.1564 | 0.2937 |
| No.25-CC2 | ①-0.9 F | NG | 36.1240 | 0.1030 | 26.6762 | 0.1885 | 35.9754 | 0.1030 | 26.6669 | 0.1885 |
| No.26-CC2 | ①-0.9 F | NG | 38.2786 | 0.0961 | 21.9691 | 0.1837 | 38.2399 | 0.0961 | 21.9642 | 0.1837 |
| No.27-CC2 | ①-0.9 F | NG | 44.5326 | 0.0894 | 13.5703 | 0.2664 | 44.5311 | 0.0894 | 13.5680 | 0.2664 |
| No.28-CC2 | ②-0.9 O | NG | 39.0126 | 0.1132 | 25.4623 | 0.1354 | 38.8641 | 0.1132 | 25.1512 | 0.1354 |
| No.29-CC2 | ②-0.9 O | NG | 40.5342 | 0.0975 | 22.4666 | 0.3042 | 40.4554 | 0.0975 | 22.3539 | 0.3042 |
| No.30-CC2 | ①-1.3 F | NG | 37.0662 | 0.1109 | 25.6351 | 0.2754 | 37.0586 | 0.1109 | 25.6348 | 0.2754 |
| No.31-CC2 | ①-1.3 F | NG | 37.3078 | 0.1069 | 24.9793 | 0.2296 | 37.3073 | 0.1069 | 24.9118 | 0.2297 |
| No.32-CC2 | ①-1.3 F | NG | 34.4628 | 0.1216 | 21.0261 | 0.2673 | 34.4550 | 0.1216 | 21.0249 | 0.2673 |
| No.33-CC2 | ①-1.3 F | NG | 37.5636 | 0.1092 | 22.8136 | 0.2404 | 37.5635 | 0.1092 | 22.7989 | 0.2404 |
| No.34-CC2 | ①-1.3 F | NG | 37.7325 | 0.1109 | 20.0425 | 0.2412 | 37.7277 | 0.1109 | 20.0422 | 0.2412 |

주) 1. Condition : ①, ②, ③ is crash test method
2. Condition 0.9 and 1.3 is vehicle weight
3. Condition F is head on collision center of facility, O is 1/4 Offset head on collision, D is 15° head on collision

Collision behavior of Crash cushion occurred very short time under 0.4 second. It

is not easy for calculation because it shows complexity behavior of the third dimensions. Therefore main design of the crash cushion is head on collision test. If passenger safety index is no difference only using of x-axis acceleration, it may easily calculate using of X-axis acceleration to apply single degree of freedom.

Figure 4 is comparing error rate, in the calculation of passenger safety using measurement data of x-axis acceleration, y-axis acceleration and yaw angular velocity from vehicle crash test during 34times crash test(All data) and calculation which uses only x-axis acceleration.




<All-Data>
X-axis acceleration = Raw Data
Y-axis acceleration = Raw Data
Yaw angular velocity = Raw Data

<X-Data>
X-axis acceleration = Raw Data
Y-axis acceleration = 0
Yaw angular velocity = 0

Figure 4. Error rate of All-data result and X-Data result

The result which is gap of passenger impact velocity(THIV) is showing that the maximum error rate of THIV is 0.90% and the average error rate is 0.14%, also gap of passenger acceleration (PHD) is maximum error rate is 1.69% and average error rate is 0.26%. There are no time error rates in THIV but PHD has the maximum time error rate 0.58% and the average time error rate 0.03%.

Therefore, hardly affected, and the passenger safety of crash cushion the results were able to confirm even if they applied three-dimensional complicated collision behavior to the single dimensions that used x-axis acceleration data only. This chapter result will use important basis to develop design method of crash cushion for using single degree of freedom.

## 5. Development of crash cushion's design method using single degree of freedom.

### 5.1 Crash analysis of crash cushion CC1

There are two ways to evaluate crash cushion CC1 presented in figure 3 showing head-on and side collision. This study doesn't use the side collision, only the data of frontal collision. Because the side clash test possesses the characteristics of scattering or behavior of crash cushion rather than passenger's safety. Therefore, this study is designed only considering the frontal collision. Crash cushion CC1 used in vehicle clash test is using pensile strength of iron bar and compressed air of rubber tire. This product is established on general concrete pavement. The size is 2,380mm(length) × 890mm(width) × 890mm(height).

Table 5 represents result of an actual vehicle crash test by TEST① in table 1. After collision the vehicle did not overturn and stood up straight on the road. Also no elements of crash cushion pierced the inner part of vehicle and there was no scattering of structural part. The result of crash test is satisfied in every item. Crash cushion CC2, CC3 can be designed by use crash test data of crash cushion CC1. Figure 6, 7 are deformation on impact force, impact energy and data of crash test result.

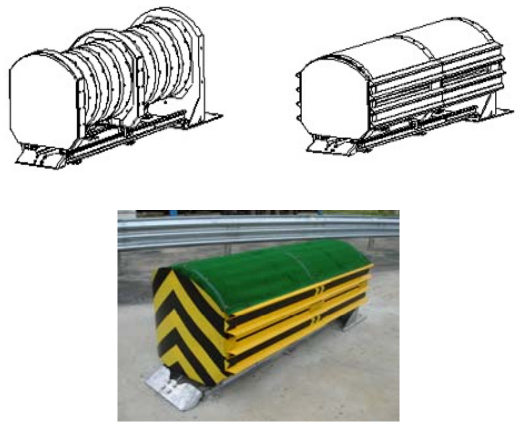


Figure 5. Whole size of crash cushion CC1

Table 5. The result of occupant risk index

| Item | Standard | Result |
|---|---|---|
| THIV | ≤ 44 km/h | 38.0 km/h |
| PHD | ≤ 20 g (g = 9.8 m/s$^2$) | 18.5 g |

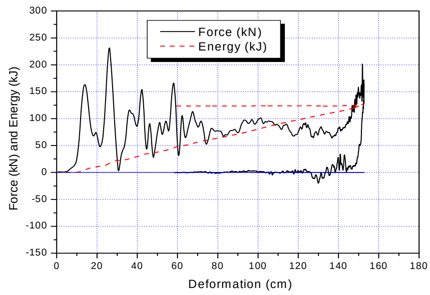



Figure 6. Transformational graph on impulsive power and energy

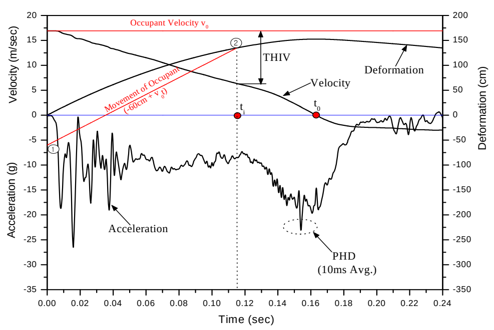



Figure 7. The graph of crash test result

5.2 Design of crash cushion using single freedom of degree

This study suggested design method uses only passenger's x-axis acceleration from the result of crash cushion verified by crash test. Design Method of this study improves confidence and accuracy using data of a vehicle crash test. But there are conditions that should precede a crash test of crash cushion CC1 and structure of crash cushion CC2, 3 is identical to CC1. Still this design is planned based on a vehicle crash test data, and so it can plan the most suitable length of product considering valid deformation distance on crash cushion CC2, 3. Therefore designed crash cushion CC2, CC3 by single degree of freedom are identical with CC1, except the length of crash cushion. It is not difficult to know the structural characteristic of crash cushion as showing graph relationship between resisting force and displacement during collision. Figure 8 is a curve of x-axis acceleration and displacement measured after collision on crash cushion CC1.

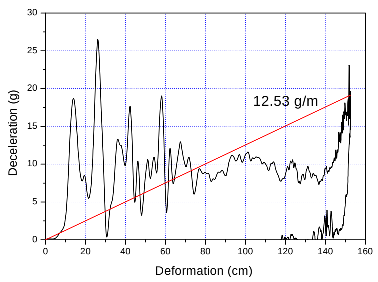



Figure 8. Acceleration-displacement by time record

An approximate value of triangular incline that has same area with integrated acceleration area before it generates elastic return can be fined 12.53g/m regardless of velocity. It represents spring constant unit mass, $k/M$.

If collision of vehicle and crash cushion sees in single degree of freedom using $M$, mass of collision vehicle and spring constant $k$, what $a/x$ is regular regardless of velocity means $k/M$ is regular.

Suppose that characteristic of crash cushion(system stiffness) is regular, the natural frequency, $\omega^2$, of single degree of freedom is regular regardless of collision velocity. The second chapter explained that if it uses single degree of freedom, displacement and acceleration of collision vehicle-crash cushion the values could be

found. That is to say that movement equation can be written formula 5, displacement, velocity and acceleration can be written formula 6, 7, 8 respectively and this formula sees that $v_0$ is velocity of initial collision, $\omega$ is $\sqrt{k/M}$ which means natural frequency of single degree of freedom. Also the maximum displacement and acceleration can be seen formula 9, 10 as a function of initial velocity, $v_0$.

As already mentioned, if the stiffness of crash cushion is decided, the value $k/M$ does not change according to collision speed. Therefore $\omega$ =( $\sqrt{k/M}$ ) is 11.087 $radians/\sec$ regardless of collision velocity. Also acceleration ( $\ddot{x}$ ) has the maximum value $v_0\omega$ at $\omega t = \pi/2$, so $t_{max}$ = 0.1sec, that time the maximum acceleration is seen formula 11.

$$\begin{aligned}|\ddot{x}_{\max}| &= v_0\omega \\ &= 11.087 rad/\sec \times v_0 km/hr \times 0.278 m/\sec/km/hr \\ &= 3.079 v_0 m/\sec^2 = 0.314 v_0 (g)\end{aligned} \tag{11}$$

In here, a unit of $v_0$ is km/hr and a unit of the maximum acceleration is gravitational acceleration(g), crash cushion $\omega^2$ =12.53g/m from formula 10, so the maximum displacement is the same with formula 12.

$$\begin{aligned}x_{\max} &= v_0/\omega \\ &= \sqrt{1/(12.53 \times 9.81)}\sec/rad \times v_0 km/hr \\ &\times 27.78 cm/\sec/km/hr = 2.505 v_0\end{aligned} \tag{12}$$

In formula 12, a unit of $x_{\max}$ is cm and $v_0$ is km/hr. Formula 11, 12 confirm that when the vehicle collide with crash cushion, if spring constant $k$ can be found, single degree of freedom' $\omega$ regardless of collision velocity is found and the maximum acceleration and displacement is calculated by formula 9, 10.

The result of crash test about crash cushion CC1 is applied to the design of single degree of freedom that is developed by this study. Using these formula, table 6 shows the maximum acceleration and valid deformation distance of crash cushion CC2 (collision velocity 80km/h) and CC3 (100km/h).

Table 6. The maximum acceleration and valid deformation distance of CC2, CC3

| Division | CC2 | CC3 |
|---|---|---|
| the maximum acceleration(g) | 25.12 | 31.4 |
| the maximum distance(cm) | 200.4 | 250.5 |

5.3 Verification of crash cushion design method using single degree of freedom

A vehicle crash test is performed by Korean guide. In here, crash cushion CC2 is the object of this test. Crash cushion CC2 is identical with crash cushion CC1 in all condition such as width, height and structure, but the length of product is different. Crash cushion CC2's length is total 4,330mm(including itself 2.8m); it applied to the maximum valid deformation distance. The length of crash cushion must be decided after consideration including the maximum valid deformation distance(2.0m), structural characteristic and thickness of crash cushion. The result of crash test about crash cushion CC2 applied to single degree of freedom has satisfied outcomes for every item. Table 7 shows the result of crash test performed by a guide.

Effectiveness of single degree of freedom for crash cushion CC2 passes 6 times crash test by a guide and finally gets a satisfied result.

Figure 9. Crash cushion CC2

Table 7. The result of crash cushion CC2 crash test

| condition | Evaluation item | | |
|---|---|---|---|
| | (THIV) | (PHD) | Behavior of crash cushion and vehicle |
| head on collision 1.3ton vehicle | 40.5 km/h | 11.6 g | No scattering of element part **good** |
| head on collision 0.9ton vehicle | 43.8 km/h | 13.0 g | No element part of separation **good** |
| head on collision ( 1/4 of vehicle's width) | 43.2 km/h | 15.3 g | stands up on the road after test **good** |
| head on collision 15° | 39.7 km/h | 13.0 g | No deformation of the vehicle's inside **good** |
| broad side collision 15° | 18.5 km/h | 7.7 g | **good** |
| broad side collision 165° | 27.2 km/h | 4.9 g | **good** |


< t = -0.1 sec >


< t = 0.0 sec >


< t = 0.1 sec >


< t = 0.2 sec >

Figure 10. Crash test of crash cushion CC2

## 6. Conclusion

Crash cushion CC1 can be verified through two times crash test according to Korea evaluation guide, but CC2 and CC3 should be done with six times crash test. CC1's successful ratio is relatively high while CC2 and CC3 is low. Also crash cushion CC1 is easy to be developed, but CC2 and CC3 is difficult. This study suggested the design method of single degree of freedom crash cushion, so that it can develop CC2 and CC3 to be rapid and inexpensive using crash cushion CC1.

The results of producing crash cushion CC2 and executing performance through a vehicle crash test is very satisfied. Judging from this, the design method of single degree of freedom is proved one of the best ways to design crash cushion. The researchers are planning to improve the single degree of freedom crash cushion more practically through performing crash test on crash cushion CC3.

## Reference

CEN(1997), Road Restraint System-part3 : Crash Cushion-Performance Classes, Impact Test Acceptance Criteria and Test Methods, prEN 1317-4, European Committee for Standardization, Lyon

Emori, I.E(1968) “Analytical Approach to Automobile Collisions” SAE Paper 680016.

H. E. Ross. Jr., D. L. Sicking, and R. A. Zimmer(1993) “Recommended Procedures for the Safety Performance Evaluation of Highway Features,” NCHRP Report 350, Transportation Research Board, Washington, D. C.

Jang Da-young, Ju Jae-woong(2006) “Act of collision considering crash cushion 's passenger safety” Korea engineering of collection of academy paper Vol.D .

Ko Man-ki, Kim Ki-dong and Jang Da-young(2003) “collision analysis and a s plan for barrier using single degree of freedom” Korea engineering of collection of academy paper, Vol.23, No.6A, pp.1193-1204; .

Michic, J. D.(1981) “Recommended Procedures for the Safety Performance Evaluation of Highway Features,” NCHRP Report 230, Transportation Research Board, Washington, D. C.

Macmillan, R. H. (1983) Dynamics of Vehicle Collisions, Proceeding of the International Association for Vehicle Design, Special Publication SP5, Channel Islands, UK.

The Ministry of Construction and Transportation (2001), “Establishment of Road Safety Facility and Management Guide- vehicle protection safety facility”.

# DEVELOPING PAKISTAN NATIONAL TRADE CORRIDOR


**AUTHOR** : **MUHAMMAD JEHANZEB MUMTAZ KHAN NIAZI**

B.Sc. (Civil), M.Sc. Transportation

Deputy Director (Planning) NATIONAL HIGHWAY AUTHORITY, PAKISTAN

**CO. AUTHOR 1**: AZEEM TAHIR, Director (PSC), NHA

**CO. AUTHOR 2**: SOICHIRO MASUDA, Advisor (JICA), NHA

**CO. AUTHOR 3**: DR. SHAHAB KHANZADA, Pavement Specialist, NHA

**CO. AUTHOR 4**: NOBORU KONDO, Senior Deputy Director, East Nippon Expressway Company Ltd, Japan

**ADDRESS**: NATIONAL HIGHWAY AUTHORITY, 27-MAUVE AREA, G-9/1, ISLAMABAD, PAKISTAN.

**E-MAIL** : jehanzebniazi@yahoo.com


---

## ABSTRACT


Pakistan is geographically located at a strategic location in South Asia. It shares South Western border with Iran, Northern border with Afghanistan and China and Eastern border with India. Arabian sea lies in south. This ideal location has merited Pakistan as most attractive for transit route to the Central Asian countries.

An efficient transport system is seen by most countries as an essential pre-condition for general economic development, and considerable resources are devoted to transport infrastructure construction and improvements. The existing transport system in Pakistan is mainly dependent on roads which suffers high economic losses and is estimated of about 4 to 6 percent of the GDP. These loses put constrains on economic growth, reduce export competitiveness and hinders social development. The main weakness of the present transport system includes high port costs, long dwell times, poor highway conditions, railways inefficiencies, custom inefficiencies, underdeveloped logistic sector.

The existing north south highways on the eastern and western banks of river Indus are serving as the backbone of transport system which connects the ports of the south to the populated cities in the north. The National Highway Authority (NHA) took the initiative under the National Trade Corridor (NTC) highway sector improvement program which comprises of three elements:

(a) construction of north-south access-controlled expressway/motorway system
(b) development of linkages of the new port of Gwadar with the NTC
(c) up-gradation of Karakoram Highway (N-35) linking China.

The main outcome of the gigantic plan is to reduce travel time to 50% (from 72 to 36 hours), travel cost by 50%, annual transportation losses from Rs 190 Billion to Rs 100 Billion and fatal accident rates by 50%. The program, having economic internal rate of return of 43%, will be implemented by 2018 with a combination of government and development banks funding.

## INTRODUCTION

Pakistan with a population of 160 Million people, is gifted with an excellent geo-strategic location with South Asia on one and Central Asia on the other side. It shares South Western border with Iran, Northern border with Afghanistan and China and Eastern border with India. Arabian Sea lies in south. This ideal location has merited Pakistan as most attractive for transit route to the Central Asian countries. For competitive trade to the Central Asian Republics (CARs), India is also dependent on this route.

The transport sector is the backbone of economy. It accounts for about 10% of Pakistan's GDP. It constitutes 12-15% of Public Sector Development Program (PSDP) and provides about 6% of total jobs (2.7 million). This sector expends 35% of the fuel energy annually. The transport system of Pakistan relies overwhelmingly on road transportation. The roads, which are about 260,000 Km in length, carry approximately 291 million tons whereas 6.4 million tons by rail. Pakistan has about 5 million vehicles on the road, growing at about 8% annually. The roads network is functioning but inadequate, inefficient & costly and causing a 4-6% loss to GDP, which translates to US $ 7-8 billion per annum.

It is well known that unless the country's infrastructure, administration and regulations are adjusted to promote modern transport and communication systems, the envisaged economic growth and export competitiveness cannot be realized. If the Pakistan does not upgrade its road infrastructure in the next five to eight years, it may lag far behind in reaping the benefits of fast changing global/regional scenario. The Vision 2030 (Transport Sector) indicates establishment of an efficient and well integrated system that will facilitate development of a competitive economy and poverty reduction, while ensuring safety in mobility. Construction of major new motorways / corridor through less populated areas to spread urbanization and the Development of international standard linkages with regional countries are inter-alia the targets fixed in vision 2030 for improving logistics/supply chain.

## THE NATIONAL HIGHWAY NETWORK

National Highway Authority (NHA) is an agency responsible for National Highways, Motorways and Strategic Roads. Length of the road network under the jurisdiction of NHA is approximately is 12000 kms and comprises primarily of strategic and principal arterial routes that serve inter-provincial long distance traffic, including important commercial cities and major freight terminals. Though, the length of National Highways is only 3.3% of the entire road network of the country but it carry more than 80% of the country's traffic.

NHA aims to construct, maintain and operate the national highways network to minimize the road transportation cost, provide driving comfort and safety to the road users and to preserve the asset investment in roads and bridges. NHA continues the active pursuit of transforming national roads to modern highways/expressways and consolidation of existing assets as well as providing linkages to remote and far flung areas from the main development stream of the country.

The Government has not adequately invested in building the road network in the past, which resulted in a huge developmental backlog at all the three levels, i.e. Federal Government, Provincial Government and Local Government roads. Unfortunately, the dilemma has not limited itself to lack of adequate investment in the past for development of road but it also engulfed the maintenance of existing road network.

National Highway N-5 which is 1819 km in length, constitutes less than 1% of country's total network, is the north-south link and is the life line of Pakistan. 80% of country's urban population lives in this corridor. It supports over 65% of trade/port traffic and serves main industrial centers which contribute 80-85% of GDP. There are different road network challenges which are given below:-

**Capacity of National Highway N-5**

- The capacity of N-5 (North-South Link) is 66,000 pcu whereas utilization upto 2008 is 50% to 93% (33,000 to 61,000 pcu). The utilization upto 2018 is estimated to 100% to 186% (66,000 to122,000 pcu)

**Quality of Existing Road Infrastructure**

- The **poor infrastructure** is costing to the economy to the tune of Rs 60-90 bil/year in extra fuel cost and subsidies on diesel. For example, the travel time between Karachi and Peshawar (1750 Km) is 72 hrs with an average commercial operating speed of 20-25kph whereas it is 24hrs for Algesiras - Paris (1855 km) with average commercial operating speed 80 to 90 kph

- The **Vehicle Operating Cost is high** because of poor quality. It is about Rs 30 bil/yr in additional road user costs. N-5, which is the main artery, about 49% length is in poor condition and require surfacing or rehabilitation

- **Poor Road Safety** is putting economic loss of Rs 75 bil/yr to the country. 7000 persons die in road accidents every year; By 2015 ~ around 14,000 fatalities; ten times higher than developed countries; (20 persons killed/year/10,000 vehicles)

**Extending Service Coverage**

- The road density of Pakistan is very low (0.32 km/square km) as compared to the Bangladesh (1.7 km/square km), Sri Lanka (1.5 km/square km), India (1.0 km/square km). At the current growth rate of the road network (4.2 percent during the past decade), it will take 50 years to arrive at the density of India.

- The new port at Gwadar has no direct link with the NTC.

- KKH that links Pakistan with China permits movement of only small trucks and is closed to traffic in winters.

**NATIONAL TRADE CORRIDOR HIGHWAY IMPROVEMENT PROGRAMME (NTCHIP)**

It has been planned to put our road infrastructure commensurate with current/futuristic needs as this critical activity is related directly to Pakistan’s overall socio-economic development. A major initiative has been launched around the National Trade Corridor (NTC) for improving transport logistics infrastructure / services by bringing the quality to international standards. The main objective is to reduce overall trade related transport logistics cost, thereby decreasing the cost of doing business & lowering indirect losses, resulting in trade competitiveness & accelerated industrialization to sustain high economic growth.


Adopting a holistic approach, the NTC improvement program (NTCIP) covers systems, procedures, & investments related to ports & shipping, energy logistics, highways improvement & trucking modernization, trade facilitation, railways restructuring / modernization and aviation / air transport modernization.

Objective of NTC Highway Improvement Program (NTCHIP) is to “improve trade flows & lower transit costs & times through sustainable delivery of efficient, safe and reliable national highways, motorways and strategic roads system thus contributing to sustained long term economic growth of Pakistan.” It consists of key sector reforms and an investment program of about US$ **4.6** billion aimed at upgrading capacity, extending the network, & modernizing national highways along NTC.

### INTELLIGENT TRANSPORT SYSTEM

Intelligent transport system (ITS) is a new transport system which is comprised of an advanced information and telecommunications network for traffic users, roads, and vehicles. ITS will result to solve the problems such as traffic accidents, and congestions. The first ever Operation Center is planned on Sehwan-Ratodero Highway N-55, which is a part of NTC, with the assistance of JICA. The project is in planning and design stage. ITS will also be established on whole NTC corridor as well as on the remaining NHA highway network in future.

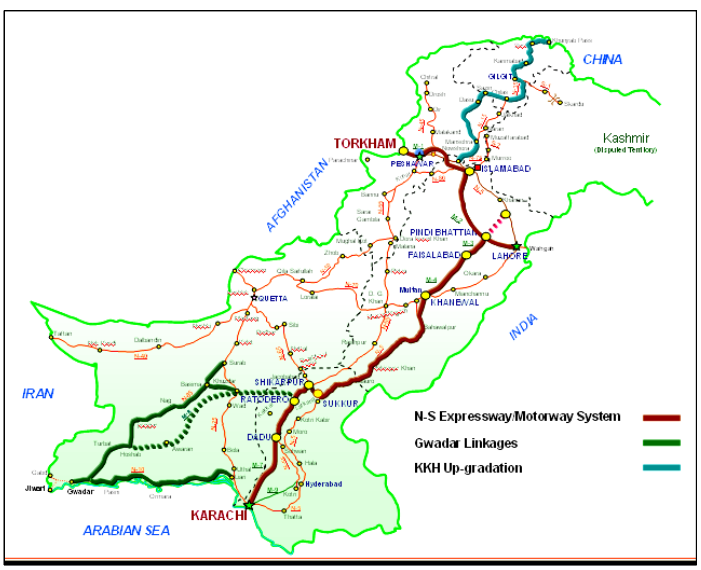
### Outcomes Influenced

NTC links the entire trade logistics network of services and infrastructure in Pakistan. Utilization of the key parts of this corridor is already more than 80% of existing capacity and the demand will double by 2015. It is estimated that inadequate performance of the transport sector costs the economy 4 to 6 % of GDP each year. In 2006, domestic transport represented (on average for 18 products) 1.29 % of the final value of the commodities against a targeted value of 0.80 % to be competitive at global level. Improved external logistics would generate savings in costs of non-factor services estimated at US$ 525 million annually. Specifically, key outcomes to be achieved, in the road sector, include:-

- 50% travel time reduction – From 48 hrs to 24 hrs between Karachi area ports & main industrial centers in Punjab; & from 72 hrs to 36 hrs between Karachi & Peshawar.
- 10% decrease in road transport costs.
- 50% reduction in road fatalities – From 20 fatalities per 10,000 vehicles to 10.

### THE NTC PROGRAMME

The NTC program is construction of about 4,268 Km roads/motorways/expressways at a cost of Rs 366 Billion by 2018 with a combination of government and development banks funding. The program includes:-



A) **Construction of an access-controlled N-S expressway/ motorway system** to provide a high-speed, safe and reliable road transport corridor. About 370 km of the motorways (M-1, M-2 & M-3) linking Peshawar with Faisalabad is already operational. It aims to complete this system expeditiously with planned financial support of various development partners. Section-wise detail is given below:-

| Route No | Section | Lanes | Km | Financing | Status/ Implementation |
|---|---|---|---|---|---|
| | **Motorways** | | | | |
| M-1 | Peshawar - Islamabad | 6 | 154 | GOP | Completed |
| M-2 | Islamabad - Pindi Bhattian | 6 | 243 | GOP | Completed |
| M-3 | Pindi Bhattian - Faisalabad | 4 | 54 | GOP | Completed |
| | **Expressways** | | | | |
| E-1 | Torkham - Peshawar | 4 | 51 | ADB | 2011-15 |
| E-2 | Peshawar-Northern Bypass | 4 | 34 | ADB | 2009-12 |
| E-3 | Pindi Bhattian - Wazirabad | 4 | 100 | WB | 2010-15 |
| E-4 | Faisalabad - Khanewal | 4 | 184 | ADB | 2009-14 |
| E-5 | Khanewal-Lodhran-Sukkur* | 4 | 485 | WB | 2010-17 |
| E-6 | Sukkur - Shikarpur- Dadu* | 4 | 231 | ADB/JICA | 2009-15 |
| E-7 | Dadu- Hub | 2 | 270 | ADB | 2012-17 |
| E-8 | Gujranwala-Wazirabad-Dina* | 4 | 100 | WB | 2012-16 |
| | **Total** | | **1,906** | | |

* Conversion of existing highways into expressways, GOP; Govt. of Pakistan, ADB; Asian Development Bank, WB; World Bank, JICA; Japan International Cooperation Agency

B) **Gwadar Linkages**

Gwadar opens up the heart of Balochistan; which comprises almost half the country's land mass. The Gwadar is not simply as a "Port" but future of Pakistan. Gwadar has already been linked with Karachi through Makran Coastal Highway (N-10). A direct link with N-S expressway/ motorway system to connect Gwadar with NTC is being developed. Plan is to build two sections of Motorway (M-8) Hoshab - Sorab and Khuzdar- Ratodero and National Highway (N-85) ie Hoshab-Sorab & Basima- Khuzdar link (N-30). Section-wise detail is given below:-

| Route No | Section | Lanes | Km | Financing | Status/ Implementation |
|---|---|---|---|---|---|
| N-10 | Gwadar - Liari | 2 | 530 | GoP | Completed |
| N-30 | Basima - Khuzdar | 2 | 110 | GoP | 2009-12 |
| N-85 | Hoshab - Basima - Sorab | 2 | 487 | GoP | 2007-12 |
| M-8 | Gwadar - Hoshab | 2 | 193 | GoP | Completed |
| M-8 | Khuzdar - Ratodero | 2 | 242 | GoP | Completed |
| | **Total** | | **1,562** | | |

C) **KKH Up-gradation**

Significant increase in trade/transit traffic from China is envisioned after opening of Gwadar Port. It has been decided to upgrade Karakoram Highway (N-35) to improve connectivity with China. Project also includes reconstruction of Sazin - Raikot Section (120 km) that will submerge in proposed Bhasha Diamir Dam reservoir.

| Route No | Section | Lanes | Km | Financing | Status/ Implementation |
|---|---|---|---|---|---|
| N-35 | Hassanabdal - Mansehra | 4 | 97 | ADB | Completed |
| N-35 | Mansehra- Sazin | 2 | 254 | GoP | 2009-14 |
| N-35 | Sazin- Raikot | 2 | 120 | GoP | 2010-15 |
| N-35 | Raikot-Khunjerab | 2 | 335 | China | 2008-13 |
| **Total** | | | **806** | | |

**THE PROGRAMME FUNDING COMMITMENTS**

The present commitments by International Financial Institutions (IFIs) including World Bank, Asian Development Bank, JICA of Japan and Development Bank of China towards the National Trade Corridor NHA's development plan provides funding to the tune of Rs 366 Billion which includes share of the World Bank; Rs 136 Billion (37%), ADB; Rs 109 Billion (30%), JICA; Rs 31 Billion (08%), China; Rs 26 billion (07%), GOP; 64 Billion (17%)

**ANALYSIS**

Countrywide traffic surveys, including 3 day manual traffic counts, origin-destination, willingness to pay surveys at 40 locations along and on the proposed alignment, Journey time surveys were conducted on all major existing proposed and alternate alignments. A countrywide model was established on the CUBE software comprising of the all the National Highway Network along with provincial highways and other relevant links based on the JICA 2005-2006 Pakistan Transport Plan Study (PTPS). Demand forecast for the collective opening year of the proposed NTC sections i.e 2015 was established and the forecast was extended to 2025.

The economic analysis for the proposed sections was computed based on the output from the traffic model comprising of Vehicle Operating Cost benefits (VOC) and Value of Time benefits (VOT). The free flow VOC model version 4 was used to establish VOC with different speeds for various categories of traffic. The collective economic internal rate of return (EIRR) for the proposed NTC links was compounded at 47% (inclusive of VOC and VOT benefits) and at 43% (with only VOC benefits).

**CONCLUSION**

Realization of Vision 2030 hinges on sustaining 7-8 % economic growth, transport infrastructure particularly NHA network is a prerequisite to sustain this growth. Any compromise on its development will be a step away from the achievement of Vision 2030.